# SYLLABUS

## ENVIRONMENTAL ENGINEERING - II

**Sub Code : 10 CV 71** **IA Marks : 25**
**No. of Lecture Hours/Week: 04** **Exam Hours : 03**
**Total No. of Lecture Hours : 52** **Exam Marks : 100**

## PART – A

**UNIT – 1:** INTRODUCTION: Necessity for sanitation, methods of domestic waste water disposal, types of sewerage systems and their suitability. Dry weather flow, factors affecting dry weather flow, flow variations and their effects on design of sewerage system. computation of design flow, estimation of storm flow, rational method and empirical formulae of design of storm water drain. Time of concentration.
06 Hours

**UNIT – 2:** DESIGN OF SEWERS: Hydraulic formulae for velocity, effects of flow variations on velocity, self cleansing and non scouring velocities, Design of hydraulic elements for circular sewers flowing full and flowing partially full (No derivations). MATERIALS OF SEWERS: Sewer materials, shapes of sewers, laying of sewers, joints and testing of sewers, ventilation and cleaning of sewers.
06 Hours

**UNIT – 3:** SEWER APPURTENANCES: Catch basins, manholes, flushing tanks, oil and grease traps, Drainage traps. Basic principles of house drainage. Typical layout plan showing house drainage connections, maintenance of house drainage.
06 Hours

**UNIT – 4:** WASTE WATER CHARACTERIZATION: Sampling, significance, techniques and frequency. Physical, Chemical and Biological characteristics, Aerobic and Anaerobic activity, CNS cycles. BOD and COD. Their significance & problems
06 Hours

## PART – B

**UNIT – 5:** DISPOSAL OF EFFLUENTS : Disposal of Effluents by dilution, self purification phenomenon. Oxygen sag curve, Zones of purification, Sewage farming, sewage sickness, Effluent Disposal standards for land, surface water & ocean. Numerical Problems on Disposal of Effluents. Streeter Phelps equation.
06 Hours

**UNIT – 6:** TREATMENT OF WASTE WATER: Flow diagram of municipal waste water treatment plant. Preliminary & Primary treatment: Screening, grit chambers, skimming tanks, primary sedimentation tanks – Design criteria & Design examples.
06 Hours

**UNIT – 7:** SECONDARY TREATMENT: Suspended growth and fixed film bioprocess. Trickling filter – theory and operation, types and designs. Activated sludge process- Principle and flow diagram, Modifications of ASP, F/M ratio. Design of ASP.
8 Hours

**UNIT – 8:** Anaerobic Sludge digestion, Sludge digestion tanks, Design of Sludge drying beds. Low cost waste treatment method. Septic tank, Oxidation Pond and Oxidation ditches – Design. Reuse and recycle of waste water.
8 Hours

**REFERENCES**

1. Manual on Waste Water Treatment : CPHEEO, Ministry of Urban Development, New Delhi.
2. Water and Wastewater Engineering Vol-II :- Fair, Geyer and Okun : John Willey Publishers, New York.
3. Waste Water Treatment, Disposal and Reuse : Metcalf and Eddy inc : Tata McGraw Hill Publications.
4. Water Technology.- Hammer and Hammer
5. Environmental Engineering: Howard S. Peavy, Donald R. Rowe, George Tchnobanoglous McGraw Hill International Edition.

## CONTENTS

## PART-A

## PART-B

# ENVIRONMENTAL ENGINEERING-II

## PART-A

## Unit-1

### INTRODUCTION:

**Necessity for sanitation**

Every community produces both liquid and solid wastes .The liquid portion –waste water– is essentially the water supply of the community after it has been fouled by a variety of uses such as spent water from bathroom kitchen, lavatory basins, house and street washings, from various industrial processes semi solid wastes of human and animal excreta, dry refuse of house and street sweepings, broken furniture, wastes from industries etc are produced daily.

If proper arrangements for the collection, treatment and disposal are not made, they will go on accumulating and create foul condition. If untreated water is accumulating, the decomposition of the organic materials it contains can lead to the production of large quantity of mal odorous gases. It also contains nutrients, which can stimulate the growth of aquatic plants and it may contain toxic compounds. Therefore in the interest of community of the city or town, it is most essential to collect, treat and dispose of all the waste products of the city in such a way that it may not cause any hazardous effects on people residing in town and environment.

Waste water engineering is defined as the branch of the environmental engineering where the basic principles of the science and engineering for the problems of the water pollution problems. The ultimate goal of the waste water management is the protection of the environmental in manner commensurate with the economic, social and political concerns.

Although the collection of stream water and drainage dates from ancient times the collection of waste water can be treated only to the early 1800s. The systematic treatment of waste water followed in the 1800s and 1900s.

**Importance of sewerage system**

One of the fundamental principles of sanitation of the community is to remove all decomposable matter, solid waste, liquid or gaseous away from the premises of dwellings as fast as possible after it is produced, to a safe place , without causing any nuisance and dispose it in a suitable manner so as to make it permanently harmless.

Sanitation though motivated primarily for meeting the ends of preventive health has come to be recognized as a way of life. In this context, development of the sanitation infrastructure of any country could possibly serve as a sensitive index of its level of prosperity. It is needless to emphasize that for attaining the goals of good sanitation, sewerage system is very essential. While provision of potable drinking water takes precedence in the order of provision of

Environmental Engineering Services, the importance of sewerage system cannot be last sight and cannot be allowed to lag behind, as all the water used by the community has to flow back as the sewage loaded with the wastes of community living , unless properly collected , treated and disposed off , this would create a serious water pollution problems.

**Definitions of some common terms used in the sanitary engineering.**

**REFUSE:**

This is the most general term to indicate the wastes which include all the rejects left as worthless, sewage, sullage – all these terms are included in this term.

**GARBAGE:**

It is a dry refuse which includes, waste papers, sweepings from streets and markets, vegetable peelings etc. The quantity of garbage per head per day amounts to be about .14 to .24 kg for Indian conditions. Garbage contains large amount of organic and putrifying matter and therefore should be removed as quickly as possible.

**RUBBISH:**

It consists of sundry solid wastes from the residencies, offices and other buildings. Broken furniture, paper, rags etc are included in this term. It is generally dry and combustible.

**SULLAGE:**

It is the discharge from the bath rooms, kitchens, wash basins etc., it does not include discharge from the lavatories , hospitals , operation theaters , slaughter houses which has a high organic matter .

**SEWAGE :**

It is a dilute mixture of the wastes of various types from the residential, public and industrial places . It includes sullage water and foul discharge from the water closets, urinals, hospitals, stables ,etc.

**STORM WATER** :

It is the surface runoff obtained during and after the rainfall which enters sewers through inlet. Storm water is not foul as sewage and hence it can be carried in the open drains and can be disposed off in the natural rivers without any difficulty.

**SANITARY SEWAGE :**

It is the sewage obtained from the residential buildings & industrial effluents establishments‘. Being extremely foul it should be carried through underground conduits.

**DOMESTIC SEWAGE:**

It is the sewage obtained from the lavatory basins, urinals &water closets of houses, offices & institutions. It is highly foul on account of night soil and urine contained in it. Night soil starts putrefying & gives offensive smell. It may contain large amount of bacteria due to the excremental wastes of patients. This sewage requires great handling &disposal.

**INDUSTRIAL SEWAGE:**

It consists of spent water from industries and commercial areas. The degree of foulness depends on the nature of the industry concerned and processes involved.

**SEWERS:**

Ewers are underground pipes which carry the sewage to a point of disposal.

**SEWERAGE:**

The entire system of collecting, carrying &disposal of sewage through sewers is known as sewerage.

**DRY WEATHER FLOW (DWF):**

Domestic sewage and industrial sewage collectively, is called as DWF. It does not contain storm water. It indicates the normal flow during dry season.

**BACTERIA:**

These are the microscopic organisms. The following are the groups of bacteria:

-Aerobic bacteria: they require oxygen &light for their survival.

-Anaerobic bacteria: they do not require free oxygen and light for survival.

- Facultative bacteria: they can exist in the presence or absence of oxygen. They grow more in absence of air.

**Invert:**

It is the lowest point of the interior of the sewer at any c/s.

**SLUDGE:**

It is the organic matter deposited in the sedimentation tank during treatment.

**Methods of domestic waste water disposal**

After the waste water is treated it is disposed in the nature in the following two principal methods

a. Disposal by Dilution where large receiving water bodies area available
b. Land disposal where sufficient land is available

The choice of method of disposal depends on many factors and is discussed later.

Sanitary engg starts at the point where water supply engg ends.

It can be classified as

- Collection works
- Treatment works
- Disposal works

The collection consists of collecting tall types of waste products of town. Refuse is collected separately. The collection works should be such that waste matters can be transported quickly and steadily to the treatment works. The system employed should be self cleaning and economical.

Treatment is required to treat the sewage before disposal so that it may not pollute the atmosphere & the water body in which it will be disposed of .The type of treatment processes depend on the nature of the waste water characteristics and hygiene, aesthetics and economical aspects.

The treated water is disposed of in various ways by irrigating fields or discharging in to natural water courses.

**Different Methods of domestic waste water disposal include (Systems of Sanitation)**

1) CONSERVENCY SYSTEM
2) WATER CARRIAGE SYSTEM

**CONSERVENCY SYSTEM**

Sometimes the system is also called as dry system. This is out of date system but is prevailing in small towns and villages. Various types of refuse and storm water are collected conveyed and disposed of separately. Garbage is collected in dustbins placed along the roads from where it is conveyed by trucks ones or twice a day to the point of disposal. all the non combustible portion of garbage such as sand dust clay etc are used for filling the low level areas to reclaim land for the future development of the town. The combustible portion of the garbage is burnt. The decaying matters are dried and disposed of by burning or the manufacture of manure.

Human excreta are collected separately in conservancy latrines.The liquid and semi liquid wastes are collected separately after removal of night soil it is taken outside the town in trucks and buried in trenches. After 2-3 years the buried night soil is converted into excellent manure. In conservancy system sullage and storm water are carried separately in closed drains to the point of disposal where they are allowed to mix with river water without treatment.

## WATER CARRIAGE SYSTEM

With development and advancement of the cities urgent need was felt to replace conservancy system with some more improved type of system in which human agency should not be used for the collection and conveyance of sewage .After large number of experiments it was found that the water is the only cheapest substance which can be easily used for the collection and conveyance of sewage. As in this system water is the main substance therefore it is called as WATER CARRIAGE SYSTEM.

In this system the excremental matter is mixed up in large quantity of water their ars taken out from the city through properly designed sewerage systems, where they are disposed of after necessary treatment in a satisfactory manner.

The sewages so formed in water carriage system consist of 99.9% of water and .1% solids .All these solids remain in suspension and do not changes the specific gravity of water therefore all the hydraulic formulae can be directly used in the design of sewerage system and treatment plants.

| CONSERVENCY SYSTEM | WATER CARRIAGE SYSTEM |
|---|---|
| 1) Very cheap in initial cost. | 1) It involves high initial cost. |
| 2) Due to foul smells from the latrines, they are to be constructed away from living room so building cannot be constructed as compact units. | 2) As there is no foul smell latrines remain clean and neat and hence are constructed with rooms, therefore buildings may be compact. |
| 3)The aesthetic appearance of the city cannot be improved | 3) Good aesthetic appearance of city can be obtained. |
| 4)For burial of excremental matter large area is required. | 4) Less area is required as compared to conservancy system. |
| 5) Excreta is not removed immediately hence its decomposition starts before removal, causing nuisance smell. | 5) Excreta are removed immediately with water, no problem of foul smell or hygienic trouble. |
| 6) This system is fully depended on human agency .In case of strike by the sweepers; there is danger of insanitary conditions in city. | 6)As no human agency is involved in this system ,there is no such problem as in case of conservancy system |

**SEWERAGE SYSTEMS:**

1) **SEPARATE SYSTEM OF SEWAGE**
2) **COMBINED SYSTEM OF SEWAGE**
3) **PARTIALLY COMINED OR PARTIALLY SEPARATE SYSTEM**

1. **SEPARATE SYSTEM OF SEWERAGE**

In this system two sets of sewers are laid .The sanitary sewage is carried through sanitary sewers while the storm sewage is carried through storm sewers. The sewage is carried to the treatment plant and storm water is disposed of to the river.

ADVANTAGES:

1) Size of the sewers are small

2) Sewage load on treatment unit is less

3) Rivers are not polluted

4) Storm water can be discharged to rivers without treatment.

DISADVANTAGE

1) Sewerage being small, difficulty in cleaning them

2) Frequent choking problem will be their

3) System proves costly as it involves two sets of sewers

4) the use of storm sewer is only partial because in dry season the will be converted in to dumping places and may get clogged.

2. **COMBINED SYSTEM OF SEWAGE**

When only one set of sewers are used to carry both sanitary sewage and surface water. This system is called combined system.

Sewage and storm water both are carried to the treatment plant through combined sewers

**ADVANTAGES:**

1) Size of the sewers being large, chocking problems are less and easy to clean.

2) It proves economical as 1 set of sewers are laid.

3) Because of dilution of sanitary sewage with storm water nuisance potential is reduced

**DIS ADVANTAGES:**

1) Size of the sewers being large, difficulty in handling and transportation.
2) Load on treatment plant is unnecessarily increased
3) It is uneconomical if pumping is needed because of large amount of combined flow.
4) Unnecessarily storm water is polluted.

**3. PARTIALLY COMINED OR PARTIALLY SEPARATE SYSTEM**

A portion of storm water during rain is allowed to enter sanitary sewer to treatment plants while the remaining storm water is carried through open drains to the point of disposal.

Advantages:-

1. The sizes of sewers are not very large as some portion of storm water is carried through open drains.
2. Combines the advantages of both the previous systems.
3. Silting problem is completely eliminated.

Disadvantages:-

1. During dry weather, the velocity of flow may be low.
2. The storm water is unnecessary put load on to the treatment plants to extend.
3. Pumping of storm water in unnecessary over-load on the pumps.

Suitable conditions for separate sewerage systems:-

A separate system would be suitable for use under the following situations:

1. Where rainfall is uneven.
2. Where sanitary sewage is to be pumped.
3. The drainage area is steep, allowing to runoff quickly.
4. Sewers are to be constructed in rocky strata. The large combined sewers would be more expensive.

Suitable conditions for combined system:-

1. Rainfall in even throughout the year.
2. Both the sanitary sewage and the storm water have to be pumped.
3. The area to be sewered is heavily built up and space for laying two sets of pipes is not enough.
4. Effective or quicker flows have to be provided.

After studying the advantages and disadvantages of both the systems, present day construction of sewers is largely confined to the separate systems except in those cities where combined system is already existing. In places where rainfall is confined to one season of the year, like India and even in temperate regions, separate system are most suitable.

Table -2.2:- Comparison of Separate and Combined systems

| Sl. no. | Separate system | Combined system |
|---|---|---|
| 1. | The quantity of sewage to be treated is less, because no treatment of storm water is done. | As the treatments of both are done, the treatment is costly. |
| 2. | In the cities of more rainfall this system is more suitable. | In the cities of less rainfall this system is suitable. |
| 3. | As two sets of sewer lines are to laid, this system is cheaper because sewage is carried in underground sewers and storm water in open drains. | Overall construction cost is higher than separate system. |
| 4. | In narrow streets, it is difficult to use this system. | It is more suitable in narrow streets. |
| 5. | Less degree of sanitation is achieved in this system, as storm water is disposed without any treatment. | High degree of sanitation is achieved in this system. |

Sources of Sewage:-

Sanitary sewage is produced from the following sources:

1. When the water is supplied by water works authorities or provided from private sources, it is used for various purposes like bathing, utensil cleaning, for flushing water closets and urinals or washing clothes or any other domestic use. The spent water for all the above needs forms the sewage.
2. Industries use the water for manufacturing various products and thus develop the sewage.
3. Water supplied to schools, cinemas, hotels, railway stations, etc., when gets used develops sewage.
4. Ground water infiltration into sewers through loose joints.
5. Unauthorized entrance of rain water in sewer lines.

Nature of Sewage:-

Sewage is a dilute mixture of the various types of wastes from the residential, public and industrial places. The characteristics and composition i.e. the nature of sewage mainly depends on this source. Sewage contains organic and inorganic matters which may be dissolved, suspension and colloidal state. Sewage also contains various types of bacteria, virus, protozoa, etc. sewage may also contain toxic or other similar materials which might have got entry from industrial discharges. Before the design of any sewage treatment plant the knowledge of the nature of sewage is essential.

Quantity of Sanitary Sewage and Storm Water:-

The determination of sanitary sewage is necessary because of the following factors which depend on this:

1. To design the sewerage schemes as well as to dispose a treated sewage efficiently.
2. The size, shape and depth of sewers depend on quantity of sewage.
3. The size of pumping unit depends on the quantity of sewage.

Estimate of Sanitary Sewage:-

Sanitary sewage is mostly the spent water of the community into sewer system with some groundwater and a fraction of the storm runoff from the area, draining into it. Before designing the sewerage system, it is essential to know the quantity of sewage that will flow through the sewer.

The sewage may be classified under two heads:

1. The sanitary sewage, and
2. Storm water

Sanitary sewage is also called as the Dry Weather Flow (D.W.F), which includes the domestic sewage obtained from residential and residential and industrials etc., and the industrial sewage or trade waste coming from manufacturing units and other concerns.

Storm water consists of runoff available from roots, yards and open spaces during rainfall.

Quantity of Sewage:-

It is usual to assume that the rate of sewage flow, including a moderate allowance for infiltration equals to average rate of water consumption which is 135 litre/ head /day according to Indian Standards. It varies widely depending on size of the town etc. this quantity is known as Dry Weather Flow (D.W.F). It is the quantity of water that flows through sewer in dry weather when no storm water is in the sewer.

Rate of flow varies throughout 24 hours and is usually the greatest in the fore-noon and very small from midnight to early morning. For determining the size of sewer, the maximum flow should be taken as three times the D.W.F.

**Design Discharge of Sanitary Sewage**

The total quantity of sewage generated per day is estimated as product of forecasted population at the end of design period considering per capita sewage generation and appropriate peak factor. The per capita sewage generation can be considered as 75 to 80% of the per capita water supplied per day. The increase in population also result in increase in per capita water demand and hence, per capita production of sewage. This increase in water demand occurs due to increase in living standards, betterment in economical condition, changes in habit of people, and enhanced demand for public utilities.

Factors affecting the quantity of sewage flow:-

The quantity of sanitary sewage is mainly affected by the following factors:

1. Population
2. Type of area
3. Rate of water supply
4. Infiltration and exfiltration

In addition to above, it may also be affected by habits of people, number of industries and water pressure etc.

Population:-

The quantity of sanitary sewage directly depends on the population. As the population increases the quantity of sanitary sewage also increases. The quantity of water supply is equal to the rate of water supply multiplied by the population. There are several methods used for forecasting the population of a community.

Type of area covered:-

The quantity of sanitary sewage also depends on the type of area as residential, industrial or commercial. The quantity of sewage developed from residential areas depend on the rate of water supply to that area, which is expressed a litres/ capita/ day and this quantity is obtained by multiplying the population with this factor.

The quantity of sewage produced by various industries depends on their various industrial processes, which is different for each industry.

Similarly the quantity of sewage obtained from commercial and public places can be determined by studying the development of other such places.

Rate of water supply:-

Truly speaking the quantity of used water discharged into a sewer system should be a little less than the amount of water originally supplied to the community. This is because of the fact that all the water supplied does not reach sewers owing to such losses as leakage in pipes or such deductions as lawn sprinkling, manufacturing processes etc. However, these losses may be largely be made up by such additions as surface drainage, groundwater infiltration, water supply from private wells etc. On an average, therefore, the quantity of sewage maybe considered to be nearly equal to the quantity of water supplied.

Groundwater infiltration and exfiltration:-

The quantity of sanitary sewage is also affected by groundwater infiltration through joints. The quantity will depend on, the nature of soil, materials of sewers, type of joints in sewer line, workmanship in laying sewers and position of underground water table.

**Infiltration** causes increase to the ―legitimate" flows in urban sewerage systems. Infiltration represents a slow response process resulting in increased flows mainly due to seasonally-elevated groundwater entering the drainage system, and primarily occurring through defects in the pipe network.

**Exfiltration** represents losses from the sewer pipe, resulting in reduced conveyance flows and is due to leaks from defects in the sewer pipe walls as well as overflow discharge into manholes, chambers and connecting surface water pipes. The physical defects are due to a combination of factors including poor construction and pipe joint fittings, root penetration, illicit connections, biochemical corrosion, soil conditions and traffic loadings as well as aggressive groundwater.

It is clear that Infiltration and Exfiltration involve flows passing through physical defects in the sewer fabric and they will often occur concurrently during fluctuations in groundwater levels, and particularly in association with wet weather events; both of which can generate locally high hydraulic gradients. Exfiltration losses are much less obvious and modest than infiltration gains, and are therefore much more difficult to identify and quantify. However, being dispersed in terms of their spatial distribution in the sewer pipe, exfiltration losses can have potentially significant risks for groundwater quality. The episodic but persistent reverse ―pumping" effect of hydraulic gain and loss will inevitably lead to long term scouring of pipe surrounds and foundations resulting in pipe collapse and even surface subsidence.

Suggested estimates for groundwater infiltration for sewers laid below ground water table are as follows:

| | Minimum | Maximum |
|---|---|---|
| Litre/ day/ hectare | 5,000 | 50,000 |
| Lpd/ km of sewer/cm dia. | 500 | 5,000 |

**Design Period**

The future period for which the provision is made in designing the capacities of the various components of the sewerage scheme is known as the design period. The design period depends upon the following:

- Ease and difficulty in expansion,
- Amount and availability of investment,
- Anticipated rate of population growth, including shifts in communities, industries and commercial investments,
- Hydraulic constraints of the systems designed, and
- Life of the material and equipment.

Following design period can be considered for different components of sewerage scheme.

1. Laterals less than 15 cm diameter : Full development
2. Trunk or main sewers : 40 to 50 years
3. Treatment Units : 15 to 20 years
4. Pumping plant : 5 to 10 years

Variations in sewage flow:-

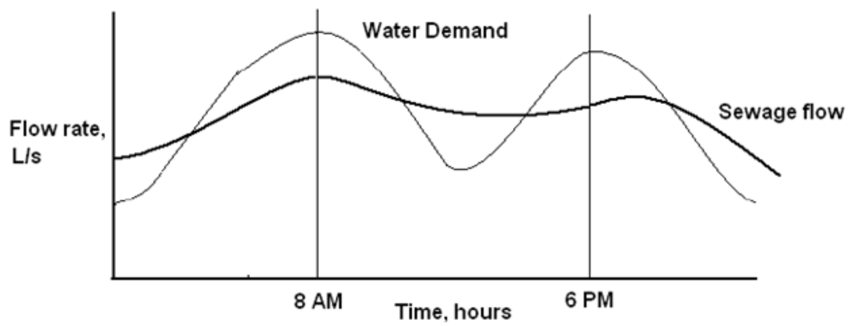
The sewage flow, like the water supply flow, is not constant in practice but varies. The fluctuation may, in a similar way, be seasonal or monthly, daily and hourly.

Variation occurs in the flow of sewage over annual average daily flow. Fluctuation in flow occurs from hour to hour and from season to season. The typical hourly variation in the sewage flow is shown in the Figure . If the flow is gauged near its origin, the peak flow will be quite pronounced. The peak will defer if the sewage has to travel long distance. This is because of the time required in collecting sufficient quantity of sewage required to fill the sewers and time required in travelling. As sewage flow in sewer lines, more and more sewage is mixed in it due to continuous increase in the area being served by the sewer line. This leads to reduction in the fluctuations in the sewage flow and the lag period goes on increasing. The magnitude of variation in the sewage quantity varies from place to place and it is very difficult to predict. For smaller township this variation will be more pronounced due to lower length and travel time before sewage reach to the main sewer and for large cities this variation will be less.

The seasonal variations are due to climatic effect, more water being used in summer than in winter. The daily fluctuations are the outcome of certain local conditions, involving habits and customs of people. Thus, in U.S.A. and other European countries, Monday is the washing day, as such, amount of sewage flow would be much greater than on any other day. In India, however, Sundays or other holidays involve activities which permit greater use of water. Hourly variations are because of varying rates of water consumption in different hours of the day.

The first peak flow generally occurs in the late morning it is usually about 200 percent of the average flow while the second peak flow generally occurs in the early evening between 6 and 9 p.m. and the minimum flow occurring during the night after twelve or early hours of the morning is generally about half of the average flow.

Importance:- the maximum and minimum rates of sewage flow are controlling factors in the design of sewers. The sewer must have ample capacity to carry the maximum flow and also to ensure sufficient velocity to produce the self cleaning which would be available in case of minimum flow.




**Typical hourly variations in sewage flow**

Effects of Flow Variation on Velocity in a Sewer

Due to variation in discharge, the depth of flow varies, and hence the hydraulic mean depth (r) varies. Due to the change in the hydraulic mean depth, the flow velocity (which depends directly on $r^{2/3}$) gets affected from time to time. It is necessary to check the sewer for maintaining a minimum velocity of about 0.45 m/s at the time of minimum flow (assumed to be 1/3$^{rd}$ of average flow). The designer should also ensure that a velocity of 0.9 m/s is developed atleast at the time of maximum flow and preferably during the average flow periods also. Moreover, care should be taken to see that at the time of maximum flow, the velocity generated does not exceed the scouring value.

Quantity of storm water flow:-

When rain falls over the ground surface, a part of it percolates into the ground, a part is evaporated in the atmosphere and the remaining part overflows as storm water. This quantity of storm water is very large as compared with sanitary sewage.

Factors affecting storm water:-

The following are factors which affect the quantity of storm water:

1. Rainfall intensity and duration.
2. Area of the catchment.
3. Slope and shape of the catchment area.
4. Nature of the soil and the degree of porosity.
5. Initial state of the catchment.

If rainfall intensity and duration is more, large will be the quantity of storm water available. If the rainfall takes place very slowly even though it continues for the whole day, the quantity of storm water available will be less.

Harder surface yield more runoff than soft, rough surfaces. Greater the catchment area greater will be the amount of storm water. Fan shaped and steep areas contribute more quantity of storm water. In addition to the above it also depends on the temperature, humidity, wind etc.

Estimate of quantity of storm water:-

Generally there are two methods by which the quantity of storm water is calculated:

1. Rational method
2. Empirical formulae method

In both the above methods, the quantity of storm water is a function of the area, the intensity of rainfall and the co-efficient of runoff.

**Rational method:-**

Runoff from an area can be determined by the Rational Method. The method gives a reasonable estimate up to a maximum area of 50 ha (0.5 $Km^2$).

**Assumptions and Limitations**

Use of the rational method includes the following assumptions and limitations:

- Precipitation is uniform over the entire basin.
- Precipitation does not vary with time or space.
- Storm duration is equal to the time of concentration.
- A design storm of a specified frequency produces a design flood of the same frequency.
- The basin area increases roughly in proportion to increases in length.
- The time of concentration is relatively short and independent of storm intensity.
- The runoff coefficient does not vary with storm intensity or antecedent soil moisture.
- Runoff is dominated by overland flow.
- Basin storage effects are negligible.
- The minimum duration to be used for computation of rainfall intensity is 10 minutes. If the time of concentration computed for the drainage area is less than 10 minutes, then 10 minutes should be adopted for rainfall intensity computations.

This method is mostly used in determining the quantity of storm water. The storm water quantity is determined by the rational formula:

$$Q = \frac{C.i.A}{360}$$

Where,

Q= quantity of storm water in $m^3$/sec

C= coefficient of runoff

i= intensity of rainfall

A= area of drainage in hectare

Runoff coefficient:-

In rational method, the value of runoff coefficient, C is required. The whole quantity of rain water that fall over the ground does not reach the sewer line. A portion of it percolates in the ground, a portion evaporates, a portion is stored in ponds and ditches and only remaining portion of rainwater reaches the sewer line. The runoff coefficient depends mainly on characteristics of ground surface as porosity, wetness, ground cover etc., which varies from 0.01 for forest or wooded area to 0.95 for a water tight roof surfaces.

As every locality consists of different types of surface area, therefore for calculating the overall runoff coefficient the following formula is used:

Runoff coefficient (overall) C $=\frac{A_1C_1+A_2C_2+A_3C_3+\cdots+A_nC_n}{A_1+A_2+A_3+\cdots+A_n}$

Where:

- $A_1$, $A_2$ ,$A_3$….are different types of area and
- $C_1$, $C_2$, $C_3$…..are their runoff coefficient respectively.

*Table* : Runoff coefficients for various surfaces.

| Type of surface | C |
|---|---|
| Watertight roofs | 0.70-0.95 |
| Asphalted cement streets | 0.85-0.90 |
| Portland cement streets | 0.80-0.95 |
| Paved driveways and walks | 0.75-0.85 |
| Gravel driveways and walks | 0.15-0.30 |
| Residential: Single-family areas | 0.30-0.50 |
| Multi units, detached | 0.40-0.60 |
| Multi units, attached | 0.60-0.75 |
| Suburban | 0.25-0.40 |
| Lawns, sandy soil | |
| 2% slope | 0.05-0.10 |
| 2-7% slope | 0.10-0.15 |
| >7% slope | 0.15-0.20 |
| Lawns, heavy soil | |
| 2% slope | 0.13-0.17 |
| 2-7% slope | 0.18-0.22 |
| >7% slope | 0.25-0.35 |

Empirical formula method:-

For determining the runoff from very large areas, generally empirical formulae are used. All the empirical formulae are applicable only under certain specific conditions such as slope of land, imperviousness, rate of rainfall etc.

1. Mc maths formula:

$$Q = \frac{C.i.A}{148.35}\sqrt[5]{\frac{S}{A}}$$

2. Burki-Zeiglar formula:

$$Q = \frac{C.i.A}{141.58}\sqrt[4]{\frac{S}{A}}$$

3. Fuller's formula:

$$Q = \frac{C.M^{0.8}}{13.23}$$

4. Talbot's formula:

$$Q = 22.4M^{1/4}$$

5. Fanning's formula:
   Q = $12.8M^{5/8}$

Where:

Q= runoff in $m^3$/sec

C= coefficient of runoff

i = intensity of rainfall in cm/hour

S = slope of area in metre per thousand metre

A = area of drainage in hectares

M = area of drainage in square km

<u>Empirical formula for rainfall intensities</u>:-

The empirical formula given by British Ministry of Health is given by:

i = $\frac{760}{t+10}$ (for storm durations of 5-20 min)

i = $\frac{1020}{t+10}$ (for storm durations of 20-100 min)

where;

i = intensity of rainfall in mm/hour

t = duration of storm in minutes

**<u>Time of concentration</u>:-**

The time taken for the maximum runoff rate to develop, is known as the time of concentration, and is equal to the time required for a drop of water to run from the farthest point of the watershed to the point for which the runoff is to be calculated.

The time of concentration, $t_c$, of a watershed is often defined to be the time required for a parcel of runoff to travel from the most hydraulically distant part of a watershed to the outlet. It is not possible to point to a particular point on a watershed and say, ―The time of concentration is measured from this point." Neither is it possible to measure the time of concentration. Instead, the concept of $t_c$ is useful for describing the time response of a watershed to a driving impulse, namely that of watershed runoff.
In the context of the rational method then, $t_c$ represents the time at which all areas of the watershed that will contribute runoff are just contributing runoff to the outlet. That is, at $t_c$, the watershed is fully contributing. We choose to use this time to select the rainfall intensity for application of the rational method. If the chosen storm duration is larger than $t_c$, then the rainfall

intensity will be less than that at $t_c$. Therefore, the peak discharge estimated using the rational method will be less than the optimal value. If the chosen storm duration is less than $t_c$, then the watershed is not fully contributing runoff to the outlet for that storm length, and the optimal value will not be realized. Therefore, we choose the storm length to be equal to $t_c$ for use in estimating peak discharges using the rational method.

The time of concentration refers to the time at which the whole area just contributes runoff to a point.

$$\mathbf{t_c = t_e + t_f}$$

Where,
$t_c$ = time of concentration
$t_e$ = time of entry to the inlet (usually taken as 5 – 10 min)
$t_f$ = time of flow in the sewer

Time of concentration is made up of inlet time (over land flow) and channel flow time.
*Time of entry (inlet time or overland flow):* is the time required for water to reach a defined channel such as a street gutter, plus the gutter flow time to the inlet.
*Channel flow time*: is the time of flow through the sewers to the point at which rate of flow is being assessed.The channel flow time can be estimated with reasonable accuracy from the hydraulic characteristics of the sewer. The channel flow time is then determined as the flow length divided by the average velocity.
The inlet time is affected by numerous factors, such as rainfall intensity, surface slope, surface roughness, flow distance, infiltration capacity, and depression storage. Because of this, accurate values are difficult to obtain. Design inlet flow times of from 5 to 30 min are used in practice.

**Estimating Time of Concentration**
There are many methods for estimating $t_c$. In fact, just about every hydrologist or engineer has a favorite method. All methods for estimating $t_c$ are empirical, that is, each is based on the analysis of one or more datasets. The methods are not, in general, based on theoretical fluid mechanics.
For application of the rational method, TxDOT recommends that $t_c$ be less than 300 minutes (5 hours) and greater than 10 minutes. Other agencies require that $t_c$ be greater than 5 minutes. The concept is that estimates of i become unacceptably large for durations less than 5 or 10 minutes. For long durations (such as longer than 300 minutes), the assumption of a relatively steady rainfall rate is less valid.

**Morgali and Linsley Method**
For small urban areas with drainage areas less than ten or twenty acres, and for which the drainage is basically planar, the method developed by Morgali and Linsley (1965) is useful. It is expressed as

$$tc = \frac{0.94(nL)^{0.6}}{i^{0.4}\,S^{0.3}}$$

where:
$t_c$ = time of concentration (min),
i = design rainfall intensity (in/hr),

n = Manning surface roughness (dimensionless),
L = length of flow (ft), and
S = slope of flow (dimensionless).

**Kirpich Method**

For small drainage basins that are dominated by channel flow, the Kirpich (1940) equation can be used. The Kirpich equation is
$tc = 0.0078(L^3/h)^{0.385}$
where:
tc = time of concentration (min),
L = length of main channel (ft), and
h = relief along main channel (ft).
The Kirpich method is limited to watershed with a drainage area of about 200 acres.

**Kerby-Hatheway Method**

For small watersheds where overland flow is an important component, but the assumptions inherent in the Morgali and Linsley approach are not appropriate, then the Kerby (1959) method can be used. The Kerby-Hatheway equation is

$tc = (0.67NL/\sqrt{S})^{0.467}$

where:
tc = time of concentration (min),
N = Kerby roughness parameter (dimensionless), and
S = overland flow slope (dimensionless).

**Problem:**

Calculate the quantity of sewage for separate and partially separate systems for a town, given the following data:

i. Area of the town – 250 hectares
ii. Intensity of rainfall – 50 mm/hr
iii. Population density – 300 persons/hectare
iv. Rate of water supply – 250 ltrs/capita/day
v. Peak factor - 2.0
vi. Surface classification:

| Type of surface | % Area | Run-off co-efficient |
|---|---|---|
| Roofs | 50% | 0.9 |
| Paved surfaces | 20% | 0.85 |
| Non paved surfaces | 30% | 0.30 |

Assume 80% of the water supplied reaches the sewer.

**Answer:**

Quantity of sewage for separate system, $Q_1 = 0.4166\ m^3/sec$
Co-efficient of run-off, C = 0.8857
Quantity of storm water partially separate system,
$Q_2 = 17.222\ m^3/sec$
Quantity of sewage for separate system, $Q = Q_1 + Q_2 = 17.639\ m^3/sec$

**Problem**

A city has a projected population of 60,000 spread over area of 50 hectare. Find the design discharge for the separate sewer line by assuming rate of water supply of 250 LPCD and out of this total supply only 75 % reaches in sewer as wastewater. Make necessary assumption whenever necessary.

**Answer:**

Given data
Q = 250 lit/capita/day
Sewage flow = 75% of water supply
= 0.75* 250 = 187.5 LPCD
Total sewage generated = 187.5*60000/(24*3600) = 130.21 lit/sec
$= 0.13\ m^3/s$
Assume peak factor = 2
Total design discharge = $0.26\ m^3/s$.

**Problem:**

A population of 40,000 is residing in a town having an area of 60 hectares, if the average coefficient of runoff for this area is 0.50 and the time of concentration of the design rain is 30 minutes. Calculate the discharge for which the sewer of a proposed combined system will be designed for the town in question.

**Answer:**

Storm discharge – 1.7 $m^3$/sec
Sewage discharge – 0.0625 $m^3$/sec
Combined discharge – 1.7625 $m^3$/sec

**Problem:**

Calculate the quantity of sewage for combined system for a town, given the following data: 1. Area of the town = 500 hectares, 2. Time of concentration = 30 mins, 3. Population density = 300 persons / hectare, 4. Rate of water supply = 135 l / capita / day, 5. Peak factor = 2.0,

| Type of surface | % Area | Run off coefficient |
|---|---|---|
| Roofs | 50 | 0.95 |
| Paved surfaces | 30 | 0.80 |
| Non paved surfaces | 20 | 0.25 |

Assume 80% of the water supplied reaches the sewer.

**Answer:**

Population P = 1,50,000
Quantity of sewage flow, $Q_1$ = 0.375 $m^3$/sec
Co-efficient of run-off, C = 0.765
Intensity of rainfall, I = 20.32 mm/hr
Quantity of storm water flow, $Q_2$ = 21.59 $m^3$/sec
Total combined flow, Q = 21.965 $m^3$/sec

**Problem:**

Design a circular stone - ware sewer with N value 0.012, running half - full to serve a town with the following data:
Estimated population = 1, 00,000
Rate of water supply = 135 lpcd
Average sewage discharge = 85% of water supply
Peak flow factor = 3
Slope of sewer = 1:300
Is the velocity developed in the sewer in self - cleansing.

**Answer:**

Quantity of sewage flow, Q = 0.398 $m^3$/sec -
Diameter of sewer, d = 0.7885 m
Velocity of flow, v = 1.63 m/sec
Velocity developed in the sewer is self cleansing

# Unit-II

## Design of sewers

After the determination of the quantity of sewage, variation in the quantity, the next step is to design the sewer section, which will be economical as well as can take the required discharge at self cleaning velocity.

## Estimate of sanitary sewage

Sanitary sewage is mostly the spent water of the community draining into the sewer system with some ground water and a fraction into the sewer system with some ground water and a fraction of the storm runoff from the area, draining into it. The sewers should be capable of receiving the expected discharge at the end of design period. The provision however should not be much in excess of the actual discharge in the early years of its use to avoid depositions in sewers. The estimate of flow therefore requires a very careful consideration and is based upon the contributory population and the per-capita flow of sewage, both the factors being guided by the design period.

## Design period

Since it is both difficult and uneconomical to augment the capacity of the system at a later date, sewers are usually designed for the maximum expected discharge to meet the requirements of the ultimate development of the area.

A design period of 30 years for all types of sewers is recommended.

The future period for which the provision is made in designing the capacities of the various components of the sewerage scheme is known as the design period. The design period depends upon the following:

- Ease and difficulty in expansion,
- Amount and availability of investment,
- Anticipated rate of population growth, including shifts in communities, industries and commercial investments,
- Hydraulic constraints of the systems designed, and
- Life of the material and equipment.

Following design period can be considered for different components of sewerage scheme.
1. Laterals less than 15 cm diameter : Full development
2. Trunk or main sewers : 40 to 50 years
3. Treatment Units : 15 to 20 years
4. Pumping plant : 5 to 10 years

## Population estimate

There are several methods for forecasting the population of a community. The most suitable approach is to base the estimation on anticipated ultimate density of population.

In case the desired information on population is not available the following densities are suggested for adoption.

Table

| Size of the town | Density of population per hectare |
|---|---|
| upto 5000 | 75-150 |
| 5000-20000 | 150-250 |
| 20000-50000 | 250-300 |
| 50000-100000 | 300-350 |
| more than 100000 | 350-1000 |

Area

The tributary area for any section under consideration need to be marked on key plan. The topography, layout of buildings, legal limitations etc., determine the tributary area draining to a sewer section. The area is to be measured from the map.

Per capita sewage flow

Although the entire spent water of a community should contribute to the total flow in a sanitary sewer, it has been observed that a small portion is lost in evaporation, seepage in ground, leakage etc. Generally 80% of the water supply may be expected to reach the sewers.The sewers should be designed for a minimum of 150 lpcd.

Ground water infiltration

Estimate of flow in sanitary sewers may include certain flows due to infiltration of ground water through joints. The quantity will depend on the workmanship in lying of sewers and the height of ground water table, the material of sewer, nature of soil etc. However the following values may be assumed.

1. 5000-50000 liters/day/hectare.

2. 500-5000 litre/day/km of sewers/cm of diameter.

Self cleansing velocity

It is necessary to maintain a minimum velocity in a sewer line to ensure that suspended solids do not deposit and cause choking troubles. Such a minimum velocity is called as self cleansing velocity. Self cleansing velocity is determined by considering the particle size and specific weight of the suspended solids in sewage.

The velocity which can cause automatic self cleansing can be found out by the following formula given by Shield:

$$V = \sqrt{[(8K/f)\ ((S_S-S)/S)\ g\ d]}$$

Where: f = Darcy‘s co-efficient of friction, 0.03

K = characteristics of solid particles

= 0.06 for organic and

= 0.04 for inorganic solids

$S_S$ = specific gravity of particles

= 2.65 for inorganic and

= 1.2 for organic solids

s = specific gravity of sewage, 1.0
G = acceleration due to gravity
D = diameter of particle

As per **Badmin Lathom's** recommendations following values of self cleansing velocities may be adopted for different sizes of sewers.

Table

| Dia. of sewers in mm | Self cleansing velocity in m/sec |
|---|---|
| 150-300 | 1.0 |
| 300-600 | 0.75 |
| >600 | 0.6 |

**Non-scouring velocity**

The smooth interior surface of a sewer pipe gets scoured due to continuous abrasion caused by the suspended solids present in sewage. The velocity of flow in sewer should not be too high, as the suspended solids will cause wear to contact surface of the pipe and erode the pipe material of sewer. This will reduce the life of the sewer. It is, therefore, necessary to limit the maximum velocity in the sewer pipe. This limiting or non-scouring velocity will mainly depend upon the material of the sewer. The permissible maximum velocity to prevent eroding is termed as non-scouring velocity and it should be limited to 3.0 m/s.

TABLE NON-SCOURING VELOCITIES.

| S.N. | Material of sewer | Non scouring velocity (cm/sec) |
|---|---|---|
| 1. | Earth channels | 60 to 120 |
| 2. | Ordinary brick-lined sewers | 150—250 |
| 3. | Cement Concrete sewers | 250—300 |
| 4. | Stone ware sewers. | 300—450 |
| 5. | Cast Iron sewer pipes | 350—450 |
| 6. | Vitrified tile and glazed bricks | 450—500 |

Empirical formulae used in design of sewers

1. Chezy's formula:

$$V = C\sqrt{R.S}$$

Where,

V = velocity of flow in m/sec
R = hydraulic mean depth in m
S = slope of the sewer = Fall of sewer/length

C = Chezy's contact.

Kutter's formula for Chezy's contact is given by:

$$C = [23 + (0.00155/S) + (1/n)] / [ 1 + (23 + (0.00155/S))(n/\sqrt{R}) ]$$

Where,

n = coefficient of roughness or rugosity factor

Table 3.3

| Sl no. | Material of sewer | Value of n |
|---|---|---|
| 1. | Brick sewers | |
| | a) Flush pointed | 0.015 |
| | b) Plastered smooth | 0.013 |
| 2. | Stoneware sewers | 0.014 |
| 3. | Cast iron | 0.013 |
| 4. | Smooth earthen channel | 0.020 |
| 5. | Rough channel | 0.030 |

2. Manning's formula:

$$V = (1/n)\, R^{2/3}\, S^{1/2}$$

(notations same as above)

3. Crimp's and Burge's formula:

$$V = 83.33\, R^{2/3}\, S^{1/2}$$

Where V, R and S have same meaning and this formula is obtained by putting n = 0.012 in Manning's formula.

4. Bazin's formula:

$$V = \frac{157.6 \quad \sqrt{R.S}}{1.81 + (K/\sqrt{R})}$$

Where K is Bazin's constant

K = 3.17 for very rough channels

= 0.11 for very smooth surfaces

5. Hazen –Williams formula:

$$V = 0.85\, C\, R^{0.6}\, S^{0.54}$$

Notations are same as previous; the value of C varies depending upon the surfaces.

Table

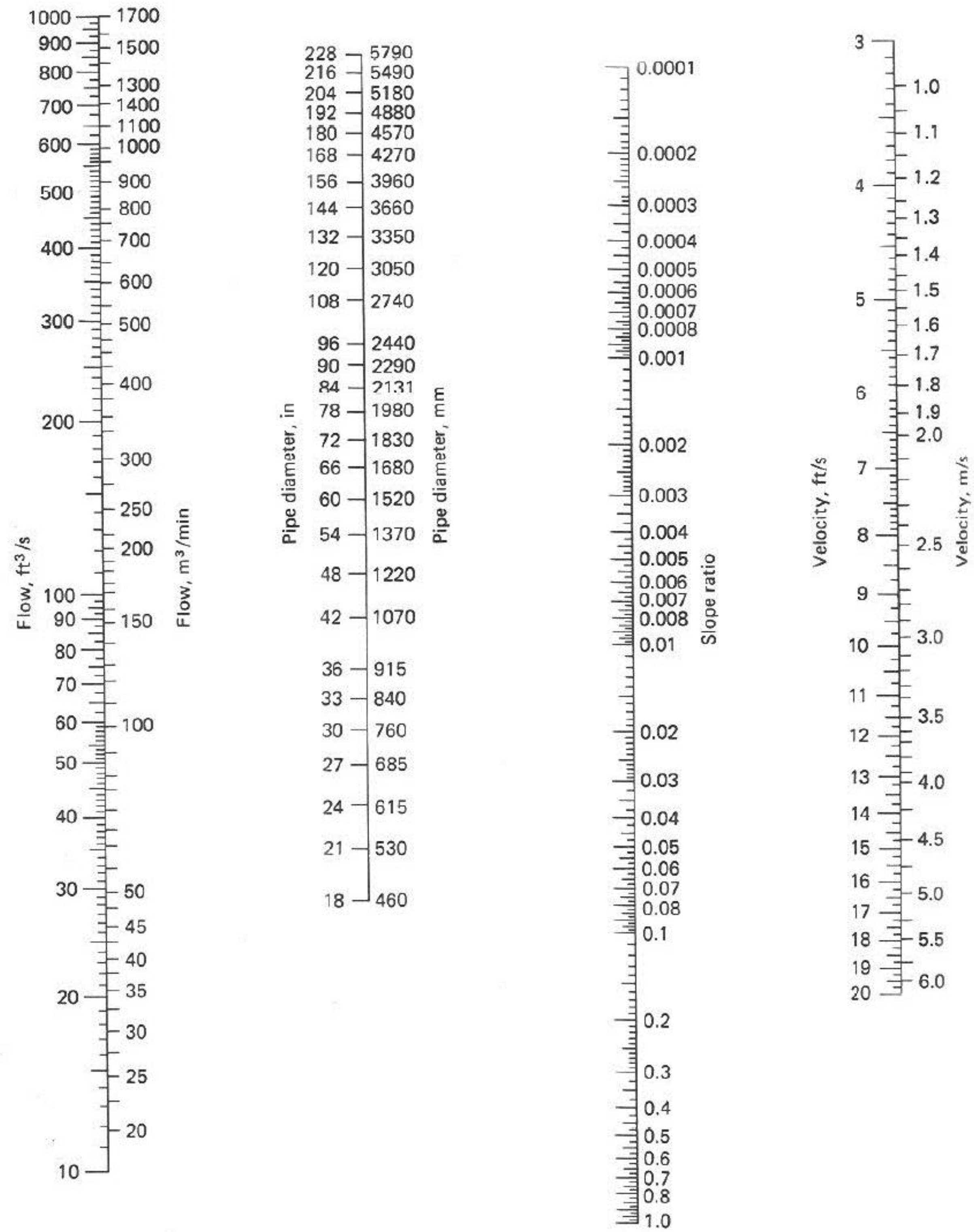
| Sl no. | Type of material | Value of C |
|---|---|---|
| 1 | Stoneware pipes in good condition | 110 |
| 2 | New cast iron | 130 |
| 3 | Very smooth surfaces | 140 |

**Design of sewers using nomograms**

While designing sewerage scheme for a town, calculations and required to done for every sewer line, to obtain the necessary slopes and sizes for given self-cleansing velocities while carrying the estimated discharges. This work involves large calculation work which is very tedious and laborious. The work of designing is simplified by the use of nomograms which have been prepared on the basis of some empirical formulae.

The nomogram shown in Fig. is based on Manning's formula in which the value of n = 0.013. The values given in the nomogram are for sewers flowing full.

For example if the required discharge of a sewer for which n = 0.013 is 200 litre/sec and the gradient is to be 0.001, a line drawn through these two values will indicate that a 650mm, sewer is needed and that the velocity will be 0.8m/sec.




**Nomogram for solution of Manning's equation for circular pipes flowing full ( n = 0.013)**

**Partial flow diagrams**

Some engineers design sewer pipes to run half-full when carrying the expected quantity. This practice has much in its favor when designing laterals and sub- mains, as a factor of safety, although it is not justified for mains and out fall sewers.

In such situations it may become necessary to determine the velocity and depth of sewage in a pipe which is flowing only partially full. Use of the following (Fig ) will allow quick computations of the hydraulic elements of partially filled circular sewers. In using the diagram it is first necessary to calculate the discharges and velocities when flowing full. Then by calculating the ratio of any two known hydraulic elements, the others can be found.

For example, if Q is the discharge when flow is full and q is the discharge when flow is partial. Then the ratio of q/Q is calculated, and using this ratio on abscissa, follow it upward till the discharge curve and from the point of intersection read the ordinate on the left hand scale, to give the (depth/diameter) ratio; then move to right hand side to intersect the velocity curve and read the abscissa to obtain v/V ratio, which will give the velocity when flow is partial.

**s/S**

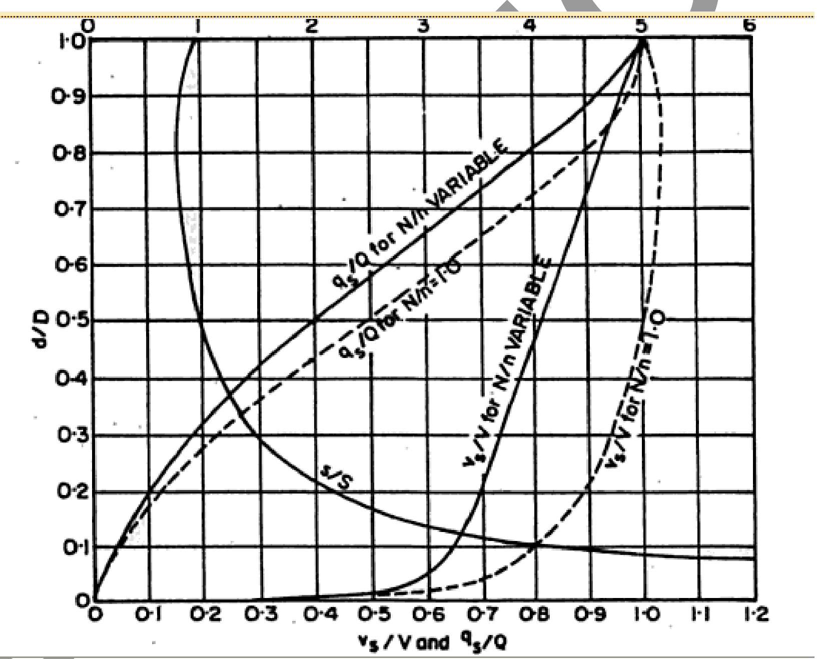



FIG. . HYDRAULIC ELEMENTS $\frac{v_s}{V}$ and $\frac{q_s}{Q}$ FOR CIRCULAR SEWERS WITH EQUAL SELF-CLEANSING PROPERTIES AT ALL DEPTHS

Note: - From the graph shown it can be concluded that:

(i) Circular sewers while running full or half full has the same hydraulic mean depth (H.M.D) and velocity of flow.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Therefore, discharging capacity of circular sewer while running half-full is exactly half the discharge of running full.

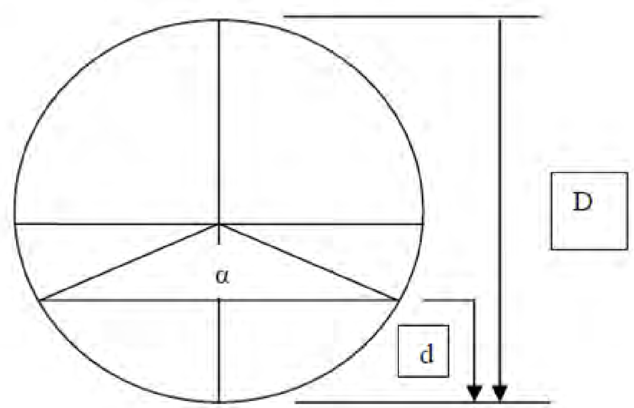
(ii) The maximum velocity is produced not when the sewer is running full, but when the depth of flow is about 0.81 times the full depth. The value of velocity at 0.81 depth is about 12.5% more than the velocity when sewer is running full. This fact is due to the reason, as the depth of flow increases beyond the mid depth, the proportionate area rises more rapidly than the proportionate wetted perimeter and this result in increasing the proportionate H.M.D. and velocities more than one.

This also raises the proportionate discharge by about 13%. The maximum discharge is reached when depth of flow is about 0.95 times the full depth.

**Hydraulic Characteristics of Circular Sewer Running Full or Partially Full**

Hydraulic Mean Depth, R = A/P

Solving for half full sewer, R = D/4




Section of a circular sewer running partially full

a) Depth at Partial flow

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

b) Therefore proportionate depth

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

c) Proportionate area

$$\frac{a}{A} = \left[\frac{\alpha}{360} - \frac{Sin\alpha}{2\pi}\right]$$

d) Proportionate perimeter: $\frac{p}{P} = \frac{\alpha}{360}$

e) Proportionate Hydraulic Mean Depth

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

f) Proportionate velocity = $\frac{v}{V} = \frac{N}{n}\frac{r^{2/3}}{R^{2/3}}$

In all above equations except ‗α' everything is constant. Hence, for different values of ‗α', all the proportionate elements can be easily calculated. These values of the hydraulic elements can be obtained from the proportionate graph prepared for different values of d/D. The value of Manning's n can be considered constant for all depths. In reality it varies with the depth of flow and it may be considered variable with depth and accordingly the hydraulic elements values can be read from the graph for different depth ratio of flow.

From the plot it is evident that the velocities in partially filled circular sewer sections can exceed those in full section and it is maximum at d/D of 0.8. Similarly, the discharge obtained is not maximum at flow full condition, but it is maximum when the depth is about 0.95 times the full depth.

The sewers flowing with depths between 50% and 80% full need not to be placed on steeper gradients to be as self cleansing as sewers flowing full. The reason is that velocity and discharge are function of tractive force intensity which depends upon friction coefficient as well as flow velocity generated by gradient of the sewer. Using subscript ‗s' denoting self cleansing equivalent to that obtained in full section, the required ratios $v_s/V$, $q_s/Q$ and $s_s/S$ can be computed.

**Numerical examples on Design of sewers**

**Problem**

The main sewer was designed for an area of 50 sq.km. Density of population of the town is 200 persons/hectare. The average flow is 250 litre/capita/day. The peak discharge is one and half times more than average flow. Rainfall equivalent of 8mm in 24 hours, all of which are runoff.

(a) What should be the capacity of the sewer in $m^3$/sec.
(b) Find the minimum velocity and gradient required to transport sewage containing coarse sand of 1mm diameter through a sewer of 35cm diameter, spec gr of particles is 2.65 and values of K = 0.06 and f = 0.03

**Solution;**

Given density of population = 200 persons/hectare

Therefore total population of the area

=(50x100) x 200 = 10,00,000 = 10 lakh

(because $1km^2$=100 hectare)

(a) Average flow of sewage = 10 x$10^5$ x 250 litre/day

= (25 x $10^7$) / (1000 x 24 x 60 x 60) = 2.893 $m^3$/sec

Max sewage flow = 1.5 x 2.893

Therefore, $Q_1$= 4.34 $m^3$/sec

Strom water flow = [(50 x 1000 x 1000) / (24 x 60 x 60)] x (8/1000)

$Q_2$ = 4.63 $m^3$/sec

Therefore total sewage flow = capacity of sewer

= 4.34 + 4.630 = 8.97 $m^3$/sec

(b) Min velocity or self cleansing velocity is given by

$$V=\sqrt{[(8k/f)\ ((S_S-S)/S)\ g.d_s]}$$

Given;

K = 0.06; f = 0.03; $S_S$ = 2.65; $d_s$ =1mm

$$V = \sqrt{[(8x0.06/0.03)\ ((2.65-1)/1)\ 9.81\ (1/1000)]}$$

= 0.51 m/sec

Hydraulic mean depth = (wetted area/wetted perimeter)

= ($\prod d^2/4$)/$\prod d$ = d/4 (for circular section)

From Manning's formula;

$V = (1/n)\ R^{2/3}\ S^{1/2}$ (assuming n = 0.012,)

$0.51 = (1/0.012) \times (0.35/4)^{2/3}\ S^{1/2}$

$S = 9.6 \times 10^{-4}$

<u>S = 1 in 1040</u>

**Problem:** Design a sewer to serve a population of 36000, the daily per capita water supply allowance being 135 litres of which 80 % finds its way into the sewer. The slope available for the sewer to be laid is 1 in 625 and the sewer should be designed to carry four times the dry weather flow when running full. What would be the velocity of flow. Take N = 0.012.

**Answer:**
**Data:**
Population = 36000
Daily per capita water supply allowance = 135 lpcd
Sewage flow = 80% of water supplied
Slope available for sewer = 1in 625
Capacity of sewer = 4 times the DWF when running full
N = 0.012
Diameter of sewer when running full = ?
Velocity of flow = ?

**Calculations:**
Quantity of sewage flow, DWF, $Q = \frac{36000 \times 135 \times 0.8}{1000 \times 24 \times 60 \times 60}$

$\mathbf{Q = 0.0145\ m3/sec}$

Capacity of sewer = 4 x DWF
= 4 x 0.045
**= 0.18 m3/sec**

Using Manning's formula,
Velocity, $V = (1/N) \times R^{2/3} \times S^{1/2}$
$V = (1/0.012) \times (d/4)^{2/3} \times (1/625)^{1/2}$
$\mathbf{V = 1.3228 \times d^{2/3}}$ **------ (1)**
Using,
$Q = A \times V$
$0.18 = (\pi d^2 /4) \times 1.3228 \times d^{2/3}$
**d = 0.5182 say 0.52 m**

Substituting the value of ‗d' in equation (1)
Velocity of flow, $V = 1.3228 \times (0.52)^{2/3}$ = **0.855 m/sec**

**Problem:**

Determine the size of an outfall sewer intended to carry the sewage flow of 650 L/s, if the average slope of the line is 1 in 1000 and the sewer is flowing half-full. Assume n = 0.015 in Manning's formula.

**Answer:**

Manning's formula, Velocity, $V = (1/N) \times R^{2/3} \times S^{1/2}$

$V = (1/0.015) \times (d/4)^{2/3} \times (1/1000)^{1/2}$

$\mathbf{V = 0.8366 \times d^{2/3}}$

Using,

$Q = A \times V$

$0.65 = (\pi d^2 /8) \times 0.8366 \times d^{2/3}$

$\mathbf{d = 1.3}$

**Problem:**
Calculate the velocity of flow in a sewer of diameter 600 mm laid on gradient of 1 in 600. What will be the discharge when running full? Assume suitable data if necessary.

**Answer:**

Assumption of Manning's rugosity Co-efficient, n = 0.012
Velocity of flow in a sewer, V = 0.9604 m/sec
Discharge when running half full, Q = 0.2715 $m^3$/sec

**Problem:**
A sewer of 300mm dia is flowing full with a slope of 1 in 300. What will be the velocity of flow and discharge when running Half-full? Is the velocity self cleansing? Assume Manning's n = 0.013.

**Answer:**
Velocity of flow, v = 0.789 m/sec
Discharge, Q = 0.02785 $m^3$/sec
Velocity developed in the sewer is self cleansing

**Problem:**
A 300 mm diameter sewer having an invert slope of 1 in 150 is flowing full. What would be the velocity of flow and discharge? Assume n = 0.013. Is the velocity self-cleansing?

**Answer:**
Discharge, Q = 0.079 $m^3$/sec
Velocity of flow, V = 1.1169 m/sec
Yes, the velocity is self cleasing

Egg Shaped Sewers

The use of egg shaped sewers is becoming obsolete these days because of their odd shape and stability. The egg shaped sewers give slightly higher velocity at small flows than the equivalent circular pipe carrying the same flow. The calculation of areas and wetted perimeters of sewers are very complicated.

For most egg shaped sewers, the following formula allows conversion to equivalent circular form. Capacities may be assumed to be equal when areas and the slope are equal.

$$d_1=0.8393d$$

Where,

$d_1$= width of egg-shaped sewers

d= diameter of circular sewer of same cross sectional area

The area A of the section is in sq m. The Perimeter P in metres and the H.M.D (R) in metres may be computed by the following formula.

$$A=9/8\, d_1^2 \quad ; \quad P= 63/16\, d_1 \quad ; R=A/P = 2/7 d_1$$

**<u>Problem.</u>**

Design an egg-shaped sewer when laid at 1 in 1000 grade, which carries a discharge of $1.5 m^3/sec$ of sewage flow.

**<u>Solution</u>**

Using formula, $A=9/8\, d_1^2$
Where A= area of sewer; $d_1$= diameter of the sewer
Assuming a self cleansing velocity is of 0.90m/sec.
Therefore Area= $\frac{\text{disharge}}{\text{velocity}} = \frac{1.5}{0.9} = 1.67 m^2$

$$1.67= \frac{9}{8} d_1^2$$

Diameter of sewer $d_1 = \frac{\sqrt{8X1.67}}{9} = 1.22m$
Vertical height = 3X radius= (3 X 1.22)/2 = 1.82m

**Construction of sewers**

Classification or types of sewers with respect to their material of construction:

The sewers may be made of:

(i) Asbestos cement

(ii) Bricks

(iii) Cast iron

(iv) Cement concrete plain or reinforced

(v) Corrugated iron sewers

(vi) Stoneware sewers

(vii) Steel sewers

(viii) Plastic sewers

(ix) Wooden sewers

**Essential requirements of a good sewer**

(a) Cost: - The cost of the material from which sewer is made, should not be high costly materials, will result in costly sewer.

(b) Durability: - Sewer should be durable and should last long.

(c) Impervious: - This is very important and essential property of a good sewer. Infiltration and exfiltration to sewer can be estimated to large extent by this property. The sewer should be impervious enough to avoid these problems. Joints in sewer line should also be impervious.

(d) Resistance to corrosion: - Because of the corrosive quality of sewage, resistance to corrosion is more important. Therefore the material used for sewer should be more corrosive resistant.

(e) Resistance to abrasion: - Since sewage contains sand and grit particles, when flowing in the high velocity, the erosion of the sewer material may takes place. Therefore the material of the sewer should be more resistant to abrasion.

(f) Weight: - To facilitate handling and transportation, the sewers should be light in weight.

(g) Strength: - Sewers are mostly laid underground. They are therefore subjected to heavy external loads. Also in soft soils there are chances of depressing the sewer at some points. This may create beam action in the sewer line. To withstand all such effects, the sewer should be made from strong material.

**Types of sewers**

Different types of sewers are discussed

1. Asbestos Cement Sewers

• These are manufactured from a mixture of asbestos fibers, silica and cement. Asbestos fibers are thoroughly mixed with cement to act as reinforcement.

• These pipes are available in size 10 to 100 cm internal diameter and length up to 4.0 m.

• These pipes can be easily assembled without skilled labour with the help of special coupling, called ‗Ring Tie Coupling' or Simplex joint.

• The pipe and joints are resistant to corrosion and the joints are flexible to permit $12^o$ deflection for curved laying.

• These pipes are used for vertical transport of water. For example, transport of rainwater from roofs in multistoried buildings, for transport of sewage to grounds, and for transport of less foul sullage i.e., wastewater from kitchen and bathroom.

Advantages

• These pipes are light in weight and hence, easy to carry and transport.

• Easy to cut and assemble without skilled labour.

• Interior is smooth (Manning n = 0.011) hence, can make excellent hydraulically efficient sewer.

Disadvantages

• These pipes are structurally not very strong.

• These are susceptible to corrosion by sulphuric acid. When bacteria produce $H_2S$, in presence of water, $H_2SO_4$ can be formed.

2. Bricks sewers: - Brick sewers are made it site. They are used for construction of large size sewers. Now a day's brick sewers are replaced by concrete sewers because lot of labour is involved in the construction of brick sewers. This material is used for construction of large size combined sewer or particularly for storm water drains. The pipes are plastered from outside to avoid entry of tree roots and ground water through brick joints. These are lined from inside with stone ware or ceramic block to make them smooth and hydraulically efficient. Lining also make the pipe resistant to corrosion.

3. Cast Iron Sewers

These pipes are stronger and capable to withstand greater tensile, compressive, as well as bending stresses. However, these are costly. Cast iron pipes are used for outfall sewers, rising mains of pumping stations, and inverted siphons, where pipes are running under pressure. These are also suitable for sewers under heavy traffic load, such as sewers below railways and highways. They are used for carried over piers in case of low lying areas. They form 100% leak proof sewer line to avoid ground water contamination. They are less resistant to corrosion; hence, generally lined from inside with cement concrete, coal tar paint, epoxy, etc. These are joined together by bell and spigot joint. IS:1536-1989 and IS:1537-1976 provides the specifications for spun and vertically cast pipes, respectively.

4. Plain Cement Concrete or Reinforced Cement Concrete
Plain cement concrete (1: 1.5: 3) pipes are available up to 0.45 m diameter and reinforcement cement pipes are available up to 1.8 m diameter. These pipes can be cast in situ or precast pipes. Precast pipes are better in quality than the cast in situ pipes. The reinforcement in these pipes can be different such as single cage reinforced pipes, used for internal pressure less than 0.8 m; double cage reinforced pipes used for both internal and external pressure greater than 0.8 m; elliptical cage reinforced pipes used for larger diameter sewers subjected to external pressure; and hume pipes with steel shells coated with concrete from inside and outside. Nominal longitudinal reinforcement of 0.25% is provided in these pipes.

Advantages of concrete pipes
• Strong in tension as well as compression.
• Resistant to erosion and abrasion.
• They can be made of any desired strength.
• Easily moulded, and can be in situ or precast pipes.
• Economical for medium and large sizes.
• These pipes are available in wide range of size and the trench can be opened and backfilled rapidly during maintenance of sewers.

Disadvantages
• These pipes can get corroded and pitted by the action of $H_2SO_4$.
• The carrying capacity of the pipe reduces with time because of corrosion.
• The pipes are susceptible to erosion by sewage containing silt and grit.
The concrete sewers can be protected internally by vitrified clay linings. With protection lining they are used for almost all the branch and main sewers. Only high alumina cement concrete should be used when pipes are exposed to corrosive liquid like sewage.

5. Corrugated iron sewers: - Corrugated iron sewers are used for storm sewers. The sewers should be protected from the effects of corrosion by galvanization or by bituminous coatings. They are made in varying metal thickness and in diameters upto 450cm.

6. Plastic sewers: - (PVC pipes) Plastic is recent material used for sewer pipes. These are used for internal drainage works in house. These are available in sizes 75 to 315 mm external diameter and used in drainage works. They offer smooth internal surface. The additional advantages they offer are resistant to corrosion, light weight of pipe, economical in laying, jointing and maintenance, the pipe is tough and rigid, and ease in fabrication and transport of these pipes.

**High Density Polythylene (HDPE) Pipes**
Use of these pipes for sewers is recent development. They are not brittle like AC pipes and other pipes and hence hard fall during loading, unloading and handling do not cause any damage to the pipes. They can be joined by welding or can be jointed with detachable joints up to 630 mm diameter (IS:4984-1987). These are commonly used for conveyance of industrial wastewater. They offer all the advantages offered by PVC pipes.

7. Steel sewers: - There sewers are used where lightness, imperviousness and resistance to high pressure are the prime requirements. There sewers are flexible and can absorb vibrations and shocks efficiently. There are mainly used for trunk or outfall sewers. Riveting should, as far as possible be avoided. These are used under the 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. They can withstand internal pressure, impact load and vibrations much better than CI pipes. They are more ductile and can withstand water hammer pressure better. These pipes cannot withstand high external load and these pipes may collapse when negative pressure is developed in pipes. They are susceptible to corrosion and are not generally used for partially flowing sewers. They are protected internally and externally against the action of corrosion.

8. Vitrified Clay or Stoneware Sewers

These pipes are used for house connections as well as lateral sewers. The size of the pipe available is 5 cm to 30 cm internal diameter with length 0.9 to 1.2 m. These pipes are rarely manufactured for diameter greater than 90 cm. These are jointed by bell and spigot flexible compression joints.

Advantages

- Resistant to corrosion, hence fit for carrying polluted water such as sewage.
- Interior surface is smooth and is hydraulically efficient.
- The pipes are highly impervious.
- Strong in compression.
- These pipes are durable and economical for small diameters.
- The pipe material does not absorb water more than 5% of their own weight, when immersed in water for 24 h.

Disadvantages

- Heavy, bulky and brittle and hence, difficult to transport.
- These pipes cannot be used as pressure pipes, because they are weak in tension.
- These require large number of joints as the individual pipe length is small.

9. Wooden sewers: - In early stages these sewers were put into use. They are difficult to construct and maintain. The life of sewers is short and they are now rarely in use.

**Shapes of Sewers**

Sewers are generally circular pipes laid below ground level, slopping continuously towards the outfall. These are designed to flow under gravity. Mostly sewers of circular shape are used in all the sewerage schemes, because of the following facts:

(i) It affords least perimeter and hence construction material required is minimum.

(ii) They are easy to construct and handle.

(iii) Since it has no corners, there are less chances of deposition of organic matters.

(iv) They possess excellent hydraulic properties.

However, sewers of non circular shapes are used for the following reasons.

(i) To develop self cleansing velocity in the sewer, when the flow is minimum.

(ii) To effect economy in the construction.

(iii) To increase the headway so that a man can enter easily for repairs, and cleaning.

Following are the non-circular shapes of sewers which are commonly used for sewers:

1. Box or rectangular sewers
2. Egg-shaped or avoid sewers
3. Basket-handle sections
4. Horse shoe sewers
5. Parabolic sewers
6. Semi-circular sewers
7. Semi-elliptical sewers
8. U-shaped sewers

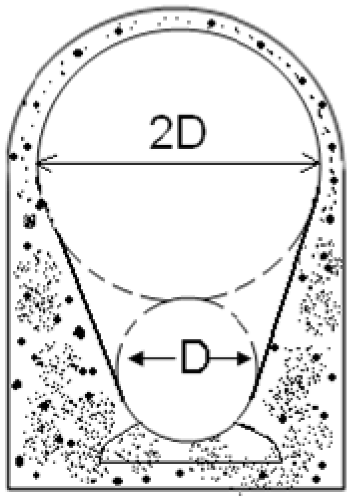
1. Box or rectangular type sewers

In olden days these sewers were constructed by laying concrete at bottom and constructing the sides with masonry. But now a day's masonry has been completely replaced by concrete. These are mainly used for out fall sewers. They have got relatively high hydraulic mean depth at large flows and therefore can have higher velocities when laid to the same slope as that of a circular or egg-shared sewer. They are therefore most suitable for large size storm sewers.

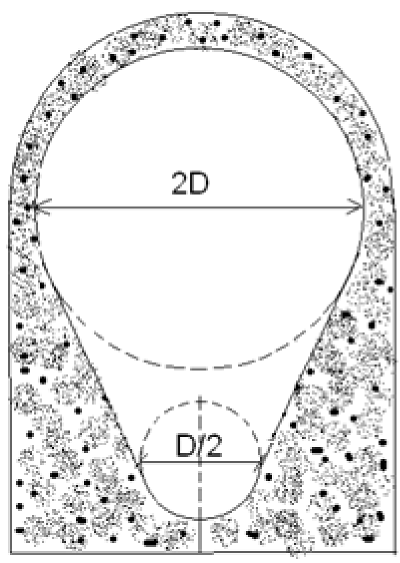
2. Egg-shared sewers

These are shown in figure. This share has got better hydraulic properties, but it is costly. Firstly due to longer perimeter more material for construction is required and secondly because of its odd shape it is difficult to construct. This sewer requires always a good foundation and proper reinforcement to make structurally stable. In India they are rarely used. They are most suitable in care of combined sewers.

The main advantage of this sewer is that it gives a slightly higher velocity during low flow, than a circular sewer of the same size.



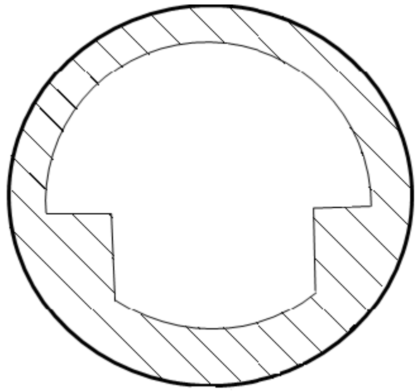

Standard Egg Shaped Sewer




New/ Modified Egg shaped Sewer

3. Basket-handle sewer.

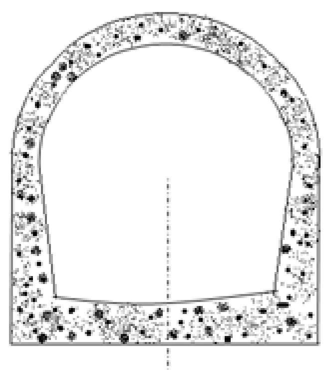
The shape of this sewer resembles the shape of a basket handle. Small discharges flow through the bottom narrower portion. During rainy days, the combined sewage flows in the full section.



Basket-Handle Section

4. Horse-shoe sewers

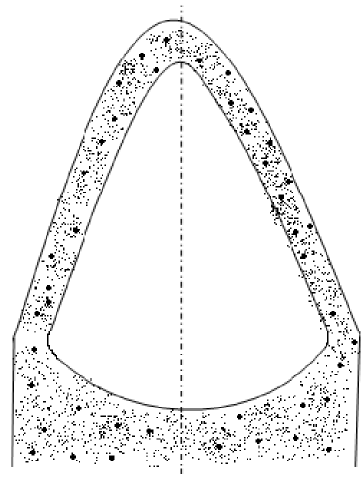
This is as shown in fig. Its top is usually semi-circular with sides inclined or vertical. The bottom may be flat, circular or paraboloid. Its height is more than width. It is mostly used for sewers in tunnels. It is used for the construction of large sewers with heavy discharged such as trunk sewers. This shape gives increased head room.



Horse shoe sewer section

5. Parabolic sewers

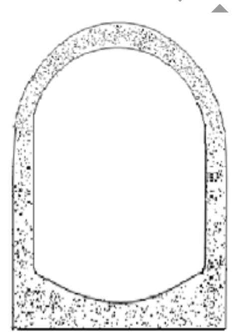
In this form of sewers, the upper arch takes the shape of parabola as shown in fig. The invert of the sewer may be flat, parabolic or elliptical. They are used for the disposal of relatively small quantities of sewage.



Parabolic section

6. Semi-circular sewers

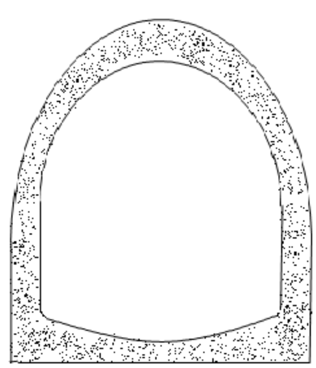
The semi-circular sewer gives a wider care at the bottom and hence, it becomes suitable for constructing large sewers with less available headroom. Now a day there are replaced by rectangular sewers.



Semi-circular Section

7. Semi-elliptical sewers

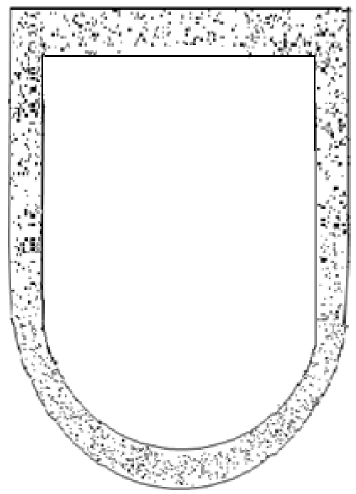
This shape of sewer is more suitable for soft soils as they are more stable. This shape is not suitable for carrying low discharges and it is normally adopted for sewers having diameter greater than 180cm or so.



Semi-elliptical section

8. U-shaped sewers

Two sections of U-shaped sewers are shown in fig. Trench provided at the bottom is called cunnette. These are easy to construct. Their invert may be flat or semi-circular. The sides are generally vertical and top may be flat or arched.



U-shaped section

**Joints in sewers**

Joints are used to john various lengths of pipes to develop a sewer line. The type of joint to be adopted depends on the pipe material, internal pressure and external loads, and many other factors. The following are the requirements of a good sewer joint.

(i) It should be water tight.

(ii) It should be easy to construct.

(iii) It should be economical.

(iv) Tree roots should not be able to penetrate the joint.

(v) It should resist to acidic, alkaline or gaseous action of sewage.

(vi) It should be flexible so that due to slight settlements in sewer line, it must not get damaged.

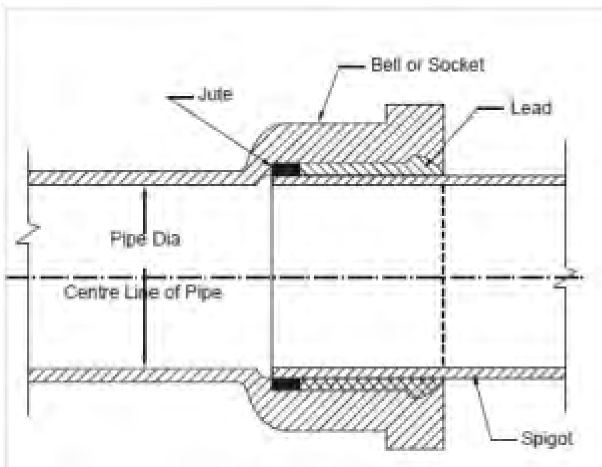
The following are the types of joints:

(1) Bandage joint

This joint is mostly adopted in concrete sewers. At the end of the pipe, a hallow is scooped out from the trench and the scooped trench is filled with cement mortar to the invert of the trench. The faces of the rises at the ends are coated with mortar and are butted against each other. Now the netting is tightly wrapped around the pipes. Additional mortar is applied around the netting, to make it firmly light.

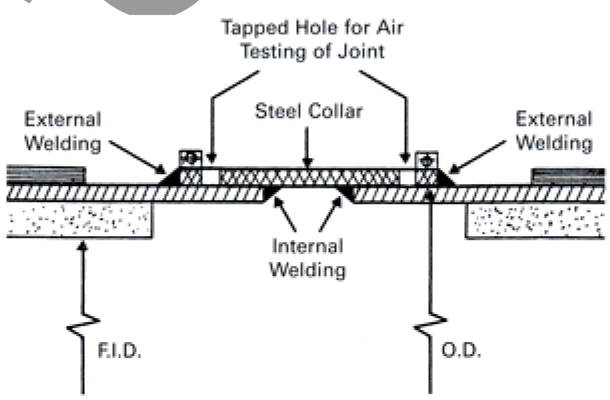
(2) Socket and spigot joints

This joint is made mainly to join cast-iron pipes and concrete pipes below the diameter of 60cm. In this type of joint, cement mortar of proportion 1:1 or 1:2 is inserted between the space of bell end and spigot end. In order to maintain the alignment of sewers, gaskets of packing pieces may be placed. The mortar is filled in the annular space formed between bell and spigot ends and the joint is finished by applying cement mortar at an angle of about 45 degree on the outer face as shown in fig.



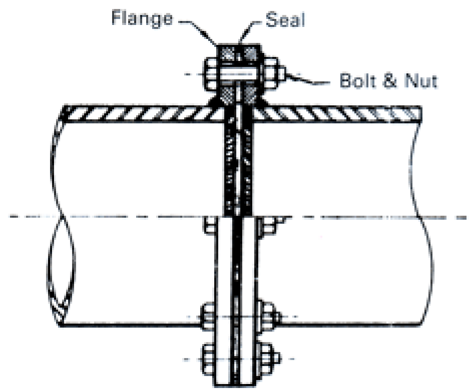

(3) Collar joints

In this type of joint, the ends of sewer are plain. The ends of sewer are placed near each other and then a collar of slightly bigger diameter is placed over the ends of sewer. The annular space between the collar and ends of sewer is filled with cement mortar. These are used for large sewers.




(4) Mechanical joints

In this type of joint, mechanical devices such as flanged rings, bolts etc. are used to join the ends of sewers together. Such type of joint is generally used for metallic sewers such as cast-iron, steel etc.




**Laying of sewers**

After the sewer plan has been approved, the next step is to set out the work. The centre line of the trench is first staked out on the ground. The centre line pegs are driven at a distance of 7.5m or 15m.

The sight rail and boning rod system is the accepted method for laying the drains accurately to the gradients, indicated on the plans. Sight rails are set at all changes of gradients and at intermediate positions, if the distance for sighting is large. The sight rails are set in such a way that, the line sighted along the top edge of the rails represents, the true fall of the sewer, this gradient is shifted below the ground level by means of a Travellor of fixed length.

Sight rails are the horizontal cross rails placed on uprights. They are usually made up of a good straight piece of timber of 15cm width and 5cm thick and length to extend over the width of the trench. Travellor or boning rod contains of a rod and T-piece. It is most important that boning rod should be cut to the exact length required; otherwise the pipes may not be laid correctly to the required grade. The boning rod may be 8cm by 4cm timber piece of required length. A T-piece of 9cm by 45cm is securely fixed by nails at top (Fig 3.3).

Since the work of laying pipes is generally started from the lower end, the sight rails will therefore, be required to fix at this point. After fixing the first set of sight rails at the tower end, a second set of sight rails is similarly set at some distance upstream side. Knowing the reduced level of invert of the sewer at the lower end and the desired gradient of the sewer line, the reduced level of invert at second set of sight rail is calculated. The depth of invert below both the sight rails should be the same to obtain the desired correct gradient, because the top of sight rails are adjusted to the correct reduced levels according to the gradient required.

**Testing of sewer line**

It is necessary to test the sewer after its laying for water tightness before backfilling of the excaved earth.

Smoke test: - This test is performed for soil pipes, vent pipes laid above ground. The test is conducted under a pressure of 2.5m 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.

Water test: - This test is performed for underground sewer pipes before back filling is done. The test should be carried out by suitably plugging the lower end of the drain and filling the system with water. A knuckle band shall be temporarily jointed at the top end and a sufficient length of vertical pipe is jointed so as to provide the required test head.

Subsidence of test water may be due to
(a) Absorption by pipes and joints
(b) Leakages at joints etc.
Any leakage if visible and defective part of work if any should be made good.

Test for straightness and obstruction: - For this test, a mirror is placed in front of one end of sewer and the image of the section is observed. If the sewer line is straight, the image should be circular. If it is not a complete circle, then it is not straight.
For testing for obstruction, by inserting a steel call at upper end and if there is no obstruction in the sewer line, the call will emerge out from the lower end.

**Ventilation of Sewer**

Sewage flowing in sewer has got lot of organic and inorganic matters present in it. Some of the matters decompose and produce gases. These gases are foul smelling, corrosive and explosive in nature. If these gases are not disposed of properly they may create a number of difficulties. They may cause air locks in sewers and affect the flow of sewage. They may prove to be dangerous for the maintenance squad working in sewers. They may also cause explosions and put the sewer line out of commission. For the disposal of these gases, ventilation of sewer line is a must.

Methods of Ventilation

Following are some of the means or fittings which help in the ventilation of sewers,

1.Laying sewer line at proper gradient.

2.Running the sewer at half full or 2/3 depth.

3.Providing manhole with gratings.

4.Proper house drainage.

5.Providing the ventilating columns or shafts.

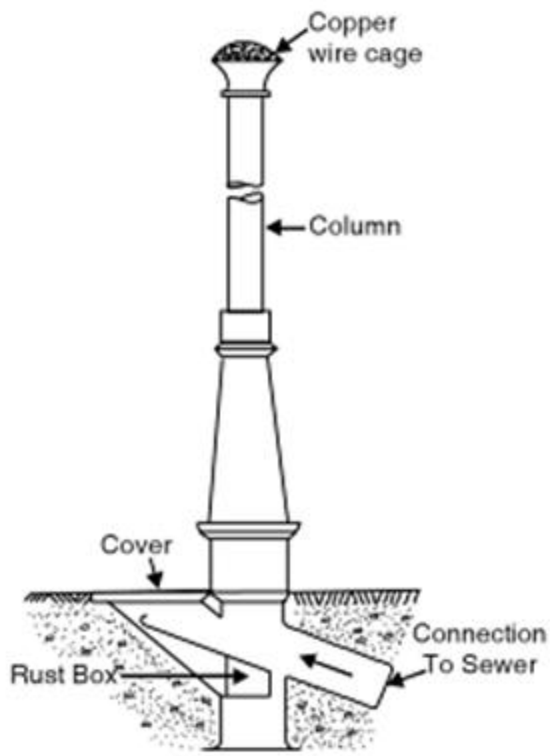



Fig. Ventilation column

# Unit-III

## SEWER APPURTENANCES

Sewage flowing in the sewer line contains a large number of impurities in the form of silt, fats, oils, rags etc. Under normal flows they are not likely to settle and choke the sewers, but during small flows self-cleansing velocity is not likely to develop and the chances of choking of the sewers are increased. Chokings have to be removed time to time, and facilities should be provided on the sewer lines for this purpose. Therefore, 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 known as sewer appurtenances. Following are the important appurtenances,

1. Manholes
2. Inlets
3. Catch basins
4. Flushing devices
5. Regulators
6. Inverted siphons
7. Grease and oil traps
8. Lamp holes
9. Leaping weirs
10. Junction chambers

## Manholes

The manholes are 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. These are fitted with suitable cast iron covers. The manholes should be installed at every points where there is a change in direction, change in pipe size, or considerable change in gradient. As far as possible sewer line between two subsequent man holes should be straight. The centre distance between manholes is less for sewers of smaller size while it may behave such a size that man can easily enter in the working chamber. The minimum size is 50cm diameter.

Table gives the spacing of manholes as recommended by the Indian Standard IS: 1742–1960.

TABLE RECOMMENDED SPACING OF MANHOLES

| *Size of sewer* | *Recommended spacing on straight reaches* |
|---|---|
| Dia. up to 0.3 m | 45 m |
| Dia. up to 0.6 m | 75 m |
| Dia. up to 0.9 m | 90 m |
| Dia. up to 1.2 m | 120 m |
| Dia. up to 1.5 m | 250 m |
| Dia. greater than 1.5 m | 300 m |

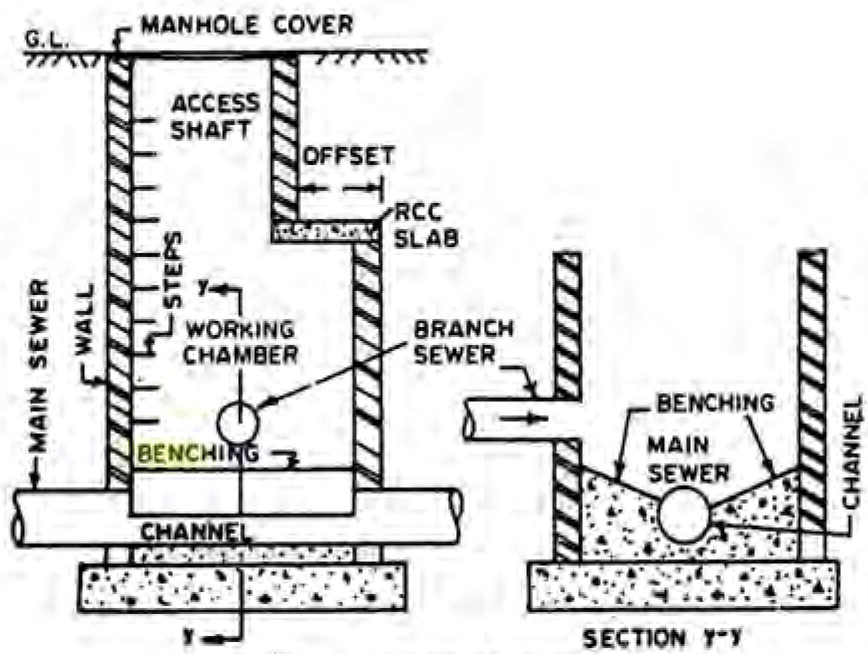



FIG. DEEP MANHOLE

**Classification of manholes :** Based upon the depth, manholes are classified as follows :

(*i*) Shallow manholes (*ii*) Normal manholes (*iii*) Deep manholes

*Shallow manholes* are the one which are about 0.75 to 0.9 m in depth. They are constructed at the start of a branch sewer. These are also known as *inspection chambers*. *Normal manholes* are those which are about 1.5 m in depth. They are constructed either in square (1 m × 1 m) or retangular (0.8 m × 1.2 m) in cross-section.

The section of such manholes is not changed with depth. It is provided with heavy cover at its top.

*Deep manholes* are those which are deeper than 1.5 m. The size of such a manhole is larger at the bottom, which is reduced at the top to reduce the size of manhole cover. The reduction in size is achieved by providing an offset of RCC, as shown in Fig. 6.6. Steps are also provided on one vertical wall of the manholes to enable the workers to go upto the bottom. A heavy manholes cover is provided at the top, with suitable C.I. frame.

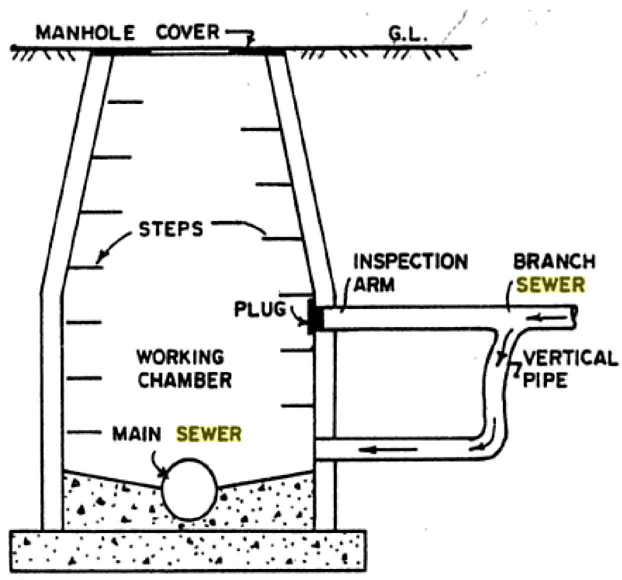
**Dimensions of manholes :** Table gives minimum internal dimensions for manhole chambers as recommended by Indian Standard IS 1742–1960.

**TABLE**

| S.N. | Depth | Min. size specified |
|---|---|---|
| 1. | 0.8 m or less | 0.75 m × 0.75 m |
| 2. | 0.8 m and 2.1 m | 1.2 m × 0.9 m |
| 3. | Greater than 2.1 m | Circular chambers with min. dia. of 1.4 m ; or retanglar chamber with min. dimension of 1.2 × 0.9 m |
| | *Min. wall thickness* | |
| | (*a*) 1.5 m depth | 20 cm |
| | (*b*) >1.5 m depth | 30 cm |

Drop Manhole

It is a measure of connecting high level branch sewer to low level main sewer. They are connected through a vertical pipe. The installation of a drop manhole becomes necessary when there is difference in levels is more than 60cm between branch sewer and the main sewer, which can be avoided by increasing the sewer grade.




Drop Manhole

Components parts of a Deep Manhole are:

i) Access shaft ii) Working chamber iii) Bottom or Invert iv) Side walls v) Steps or ladder vi) Top cover

Inlets

These are meant to admit the surface runoff to the sewers and form a very important part of the system. Their location and design should therefore be given careful consideration.

Storm water inlets may be categorised under three major groups viz, curb inlets, gutter inlets, and combination inlets, each being either depressed or flush depending upon their elevation with reference to the pavement surface.

The actual structure of an inlet is usually made of brick work. Normally cast iron gratings gratings confirming to IS:961 shall be used. The clear opening shall not be more than 25mm. The connecting pipe from the street inlet to the main street sewer should not be less than 200mm dia. and should have sufficient slope.

Maximum spacing of inlets would depend upon various conditions of road surface, size and type of inlet and rainfall. A maximum spacing of 30m is recommended.

Curb Inlets

Curb inlets are vertical openings in the road curbs through which the storm water flows and are preferred where heavy traffic is anticipated.

Gutter Inlets

Gutter inlet is sometimes called horizontal inlet also. This inlet is constructed in road gutter and storm water enters directly into it through horizontal grating provided at the top of the inlet. Such inlets are suitable for roads having steep slope, because its capacity to handle storm water is quite large. Fig.4.1d

Combination Inlets

These are composed of a curb and gutter inlet acting as a single unit. Normally, the gutter inlet is placed right in front of the curb inlet but it may be displaced in an overlapping end-to-end position.

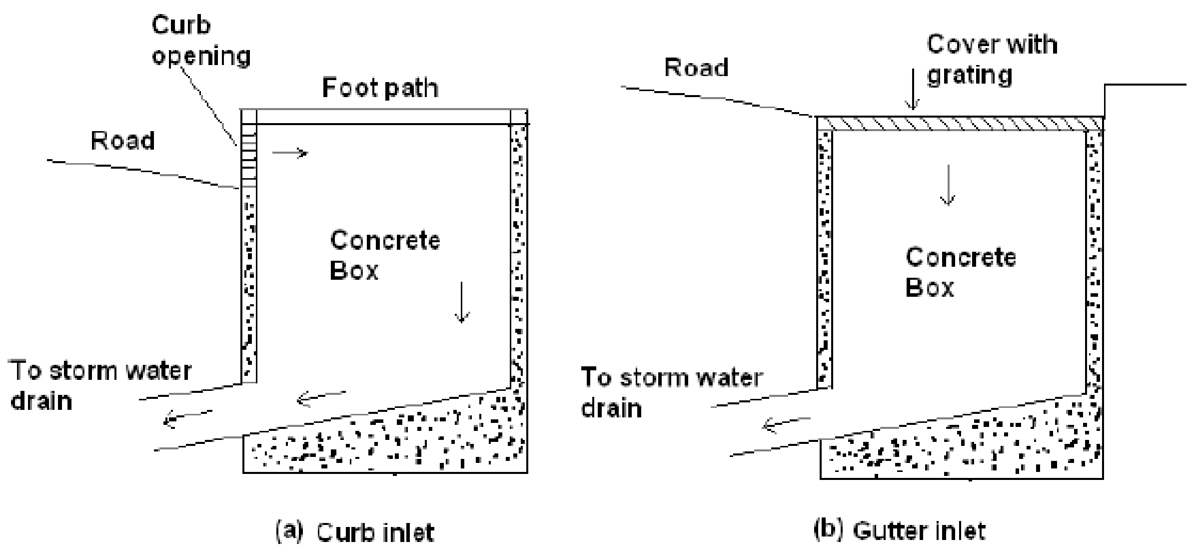
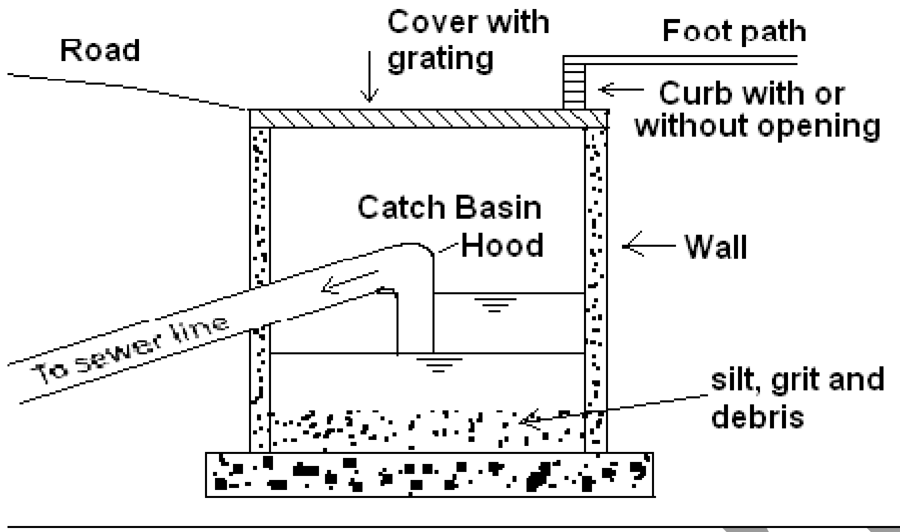



Figure (a) Curb inlet and (b) Gutter inlet

Catch Basins

Catch basins are the structures of pucca chamber and a stout cover. They are meant for the retention of suspended grit, sludge and other heavy debris and floating rubbish from rain water which otherwise might have entered and cause choking problems. The outlet pipe from the catch basin may be submerged in order to prevent the escape of odours from the sewer and provision that also causes retention of floating matter. Their use is not recommended since they are more of a nuisance and a source of mosquito breeding apart from posing substantial maintenance problems.




Catch Basins

## Flushing devices

Flushing tanks are provided to flush the sewers. They are seldom used. At such places where self cleansing velocity is not developed or when the ground is flat and it is not possible to lay the sewer lines at designed gradients, flushing tanks required to flush the sewer line. They are installed at suitable intervals to clean the sewers of choking and obstructions.

It resembles a manhole but it is equipped with a siphon at the bottom. This is called the automatic flushing tank in which the water is automatically released from the tank at suitable intervals which may be water supply pipe tap.

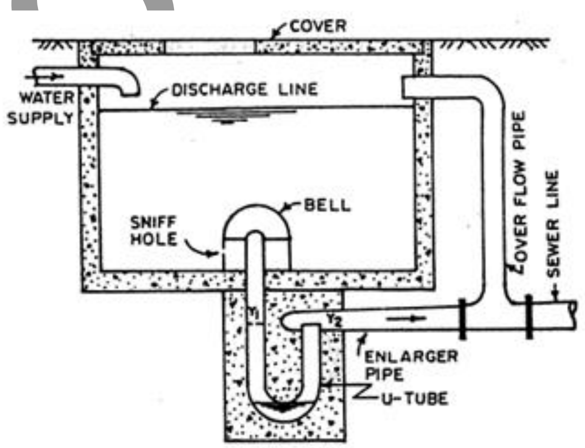


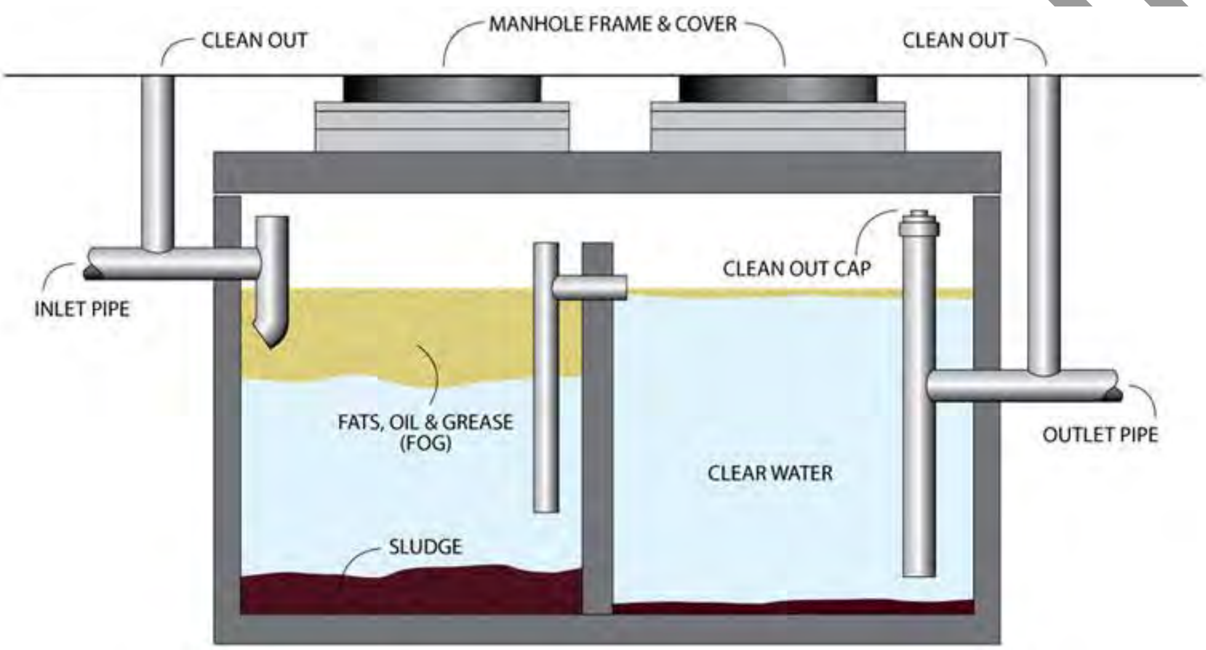

FIG. AUTOMATIC FLUSHING TANK

## Grease and Oil traps

The sewage from kitchens of hotels and restaurants and industries contains oil and grease and fats. If these oils and greases are not removed from the sewage they will stick to the interior surface of the sewer and clogging. Sewage from garages, particularly from floor drains and wash racks, contains oil, mud and sand.

The principle, on which oil and grease trap work, is since oil and grease being lighter than water float on the surface of sewage, and the outlet is provided well below the surface so the water is excluded from oil and grease.

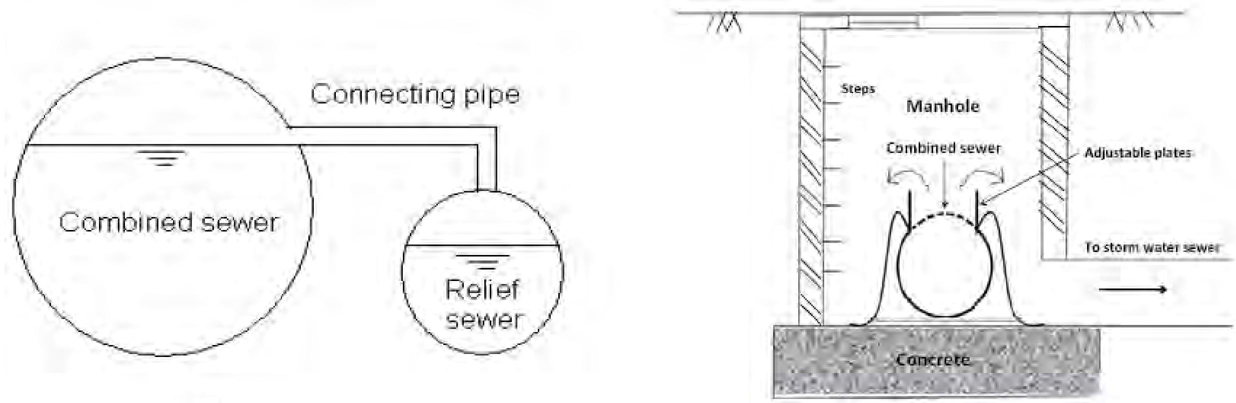
If silt also has to be excluded, it is done by providing outlet at top. The silt settles at bottom and silt free water can be drained through outlet.




Grease and Oil traps

Regulators

A Regulator is a device that diverts sewage flow from one sewer into another. The regulator usually goes into action when the sewage flow reaches a predetermined amount. It may then divert all the sewage or only that part above the predetermined flow at which it begins to function. Regulators are mostly used where combined sewers discharge into interceptors. The interceptor take the dry-weather flow, but the storm water is diverted into a sewer which flows to the nearest water course.



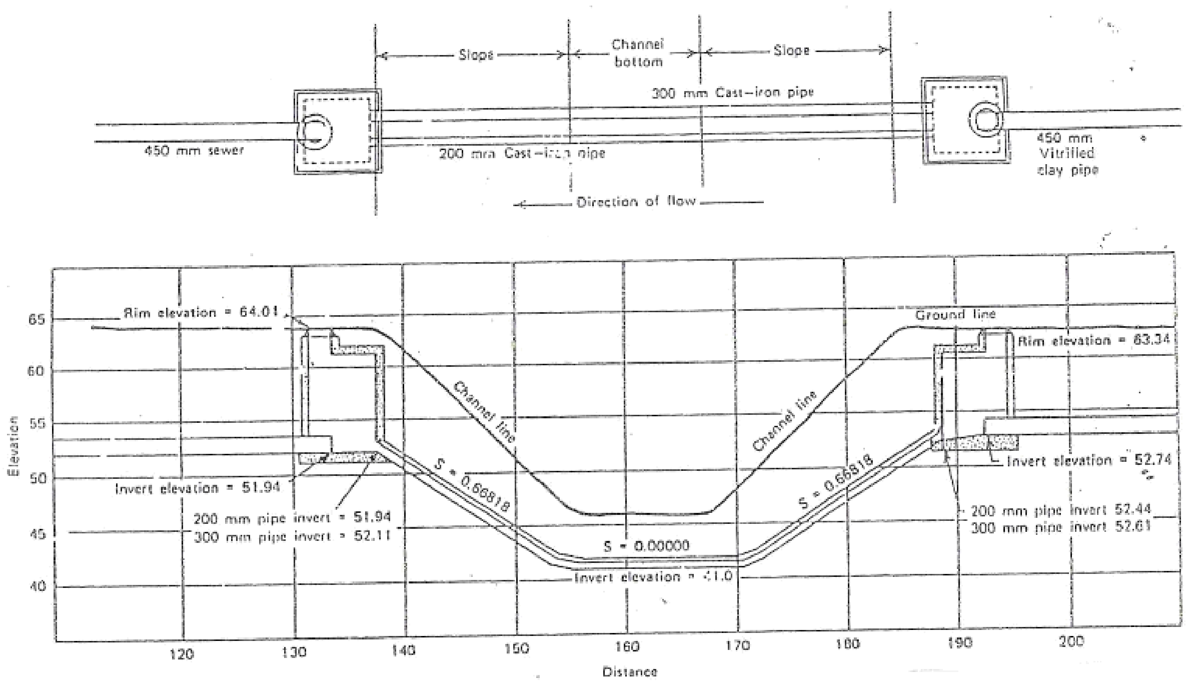

(a) (b)

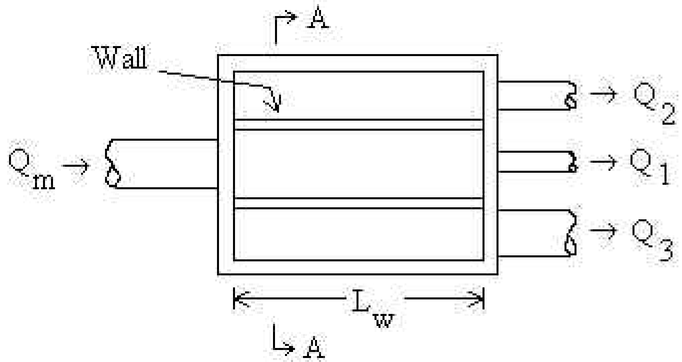
Figure (a) Side flow weir (b) Overflow weir arrangement

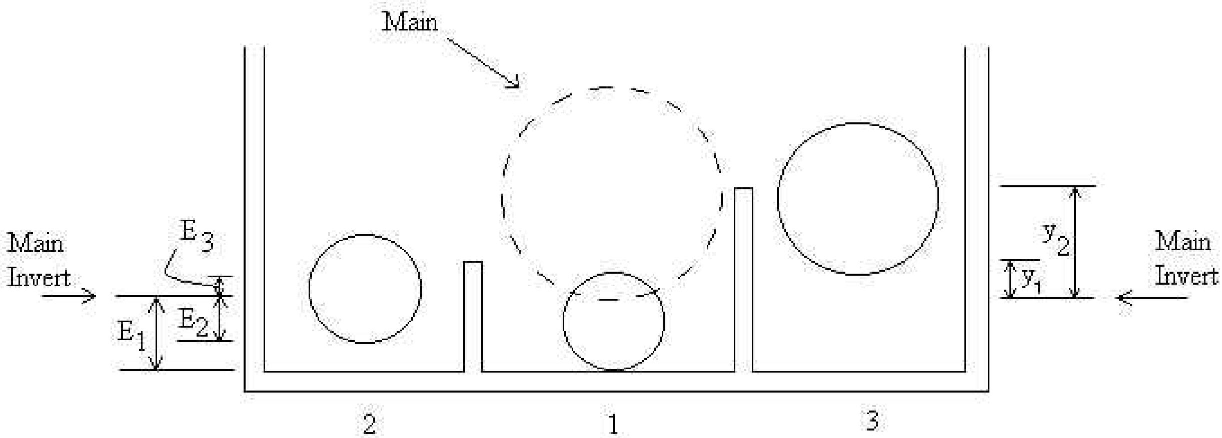
Inverted siphons

The true siphon is a sewer that runs full having its flow line above the hydraulic grade line. The pressure in the sewer line will be greater than atmospheric pressure. On the other hand inverted siphon or depressed sewer that runs under full gravity flow, at a pressure above the atmospheric pressure, the profile being depressed below the hydraulic grade line. (fig 4.1h)

Inverted siphons are used to pass sewer lines, under obstacles such as buried pipes, subways and stream beds. The pipe used must be above to withstand internal pressure. As high velocities as possible should be maintained, with 0.9m/sec minimum. Inverted siphon may consist of one, two or more pipes depending upon the amount of flow and head available.





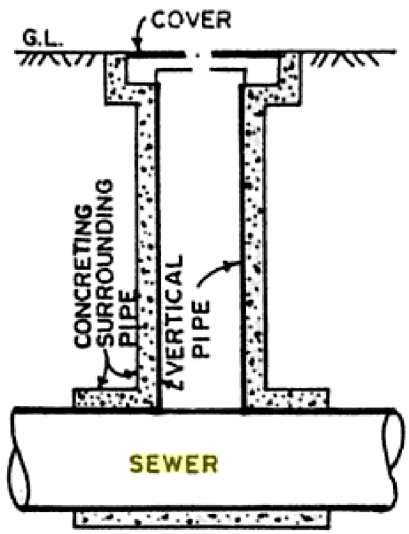

## Lamp Holes

Lamp holes consists of a vertical A.C. pipe of 22.5cm diameter and connected to the main sewer with the help of a Tee lamp holes are provided for the following purposes

(1)Distance between the man holes is usually large and it is not possible to approach full length from both the man holes. Lampholes are provided between the man holes. Sewer line is examined by lowering an electric lamp in the lamp hole and observing the light from the adjacent man holes. If light is visible, that it indicates that the length of the sewer is unobstructed.

(2)They may be used under certain circumstances for flushing the sewer line.

(3)By providing a perforated cover, the ventilation of the sewer can be achieved.




Lamp Holes

Leaping Weir

It consists of a gap in the invert of the combined sewer. The intercepting sewer runs at a lower level and at the right angles to the combined sewer.

When the flow in the combined sewer is only D.W.F the whole flow directly falls into the intercepting sewer through the gap. But as the discharged exceed the predetermined amount, the excess sewage jumps or the leap across the gap and is carried to the natural stream or river through storm water sewer. Leap weir is show in fig.

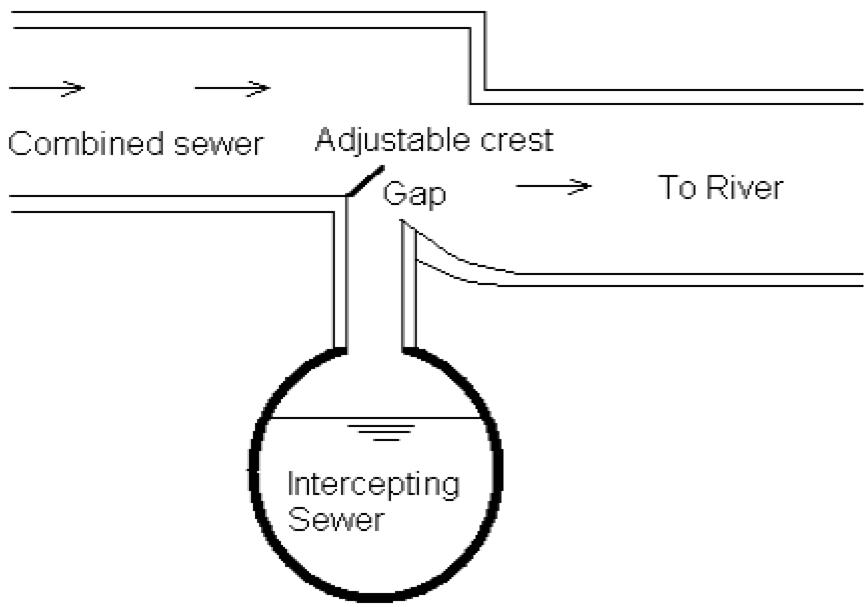



Figure Leaping weir with adjustable crest

Junction chamber

It is the chamber constructed to facilitate the junction of two or more sewers, which are large enough for a man to enter. The junction of small sewers is made in manholes. The junction of a small sewer with large one large enough to be entered is usually a Y branch or slant penetrating the sewer wall. Where two large sewers join, either a bell mouthed or a flat-topped junction may be used. In each case care should be taken that there is no decrease in velocity and a minimum of disturbance in either sewer or the junction which would cause eddies and deposition of solids. The normal flow lines of the intersecting sewers should coincide at the junction.

**Traps - Types and Uses**

A trap is a device which is used to prevent sewer gases from entering the buildings. The traps are located below or within a plumbing fixture and retains small amount of water. The retaining water creates a water seal which stops foul gases going back to the building from drain pipes. Therefore all plumbing fixtures such as sinks, washbasins, bathtubs and toilets etc. are equipped with traps. This article tells you the features of traps, various types of traps and water seal.

A trap has following features.

- It may be manufactured as an integral trap with the appliance as in some models of European WC, or it may be a separate fitting called an attached trap, which is connected to waste or foul water outlet of appliances.
- The traps should be of a self-cleansing pattern.

- Traps for use in domestic waste should be convenient for cleaning.
- A good trap should maintain an efficient water seal under all conditions of flow.

**Various Types of Traps**

**1. Gully Trap:**

These traps are constructed outside the building to carry waste water discharge from washbasin, sinks, bathroom etc. and are connected to the nearest building drain/sewer so that foul gases from sewer do not come to the house. These are deep seal traps, the depth of water seal should be 50 mm minimum. It also prevents the entry of cockroach and other insects from sewer line to waste pipes carrying waste water.

**2. P. Trap:**

This trap is used with Indian water closet (ORISSA Pattern). The traps are made from cast iron or UPV sheet. This trap also has water seal and prevents entry of foul gases to the house.

**3. S. Trap:**

This trap is similar to P. trap and is used for fixing water closets in toilets. The only difference between P trap and S trap is that P. trap is used for outlet through the wall whereas S. trap is used for outlet through the floor.

**4. Floor Trap or Nahini Trap:**

This trap is provided in the floor to collect waste water from washbasin, shower, sink and bathroom etc. These are available in cast iron or UPVC material and have removable grating (JALI) on the top of the trap. The minimum depth of water seal should be 50 mm.


**5. Intercepting Trap:**
This trap is provided at the last main hole of building sewerage to prevent entry of foul gases from public sewer to building sewer. It has a deep-water seal of 100 mm.

**6. Grease Trap:**
This trap is a device to collect the grease contents of waste and can be cleaned from the surface. This is generally used in food processing unit.

**7. Bottle Trap:**
This trap is used below washbasin and sinks to prevent entry of foul gases.

**8. Q Trap:**

This trap is used in toilet under water closet. It is almost similar to S trap and is used in upper storey other than ground floor.

**Water Seal**

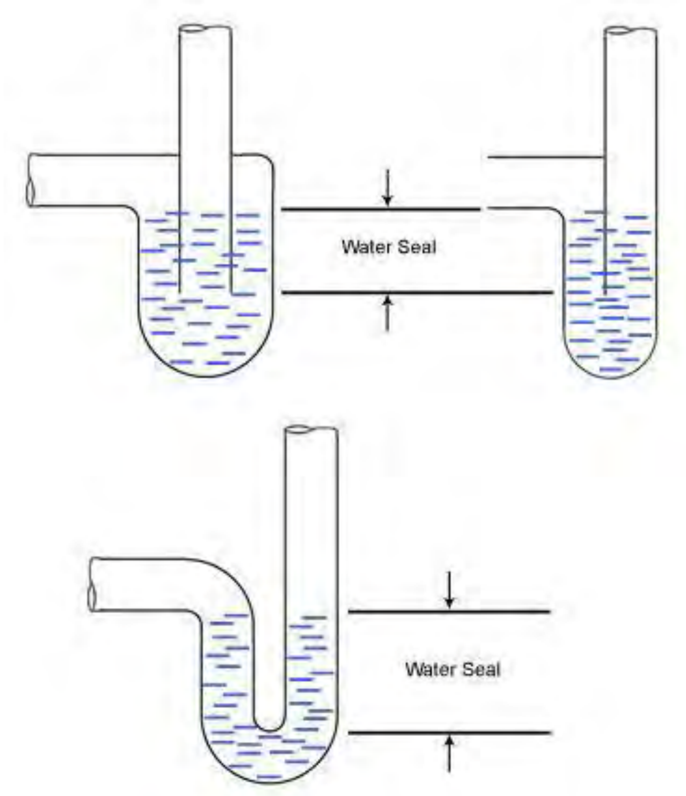
Water seal in a trap is the depth of water which should be removed from a fully charge trap before gases at atmospheric pressure may pass from the waste pipe through trap into a building. The tape is useless unless they retain their seals at all times. The seal may be broken due to air compressor, momentum and evaporation. The trap in fittings in range is liable to siphonic action and each trap should be ventilated.

The depth of water seal in various sanitary appliances is given below.

- Water closet 50 mm
- Floor trap 50 mm

Other fixtures are directly connected to the stack through branch waste pipe of diameter = 75 mm and 40 mm

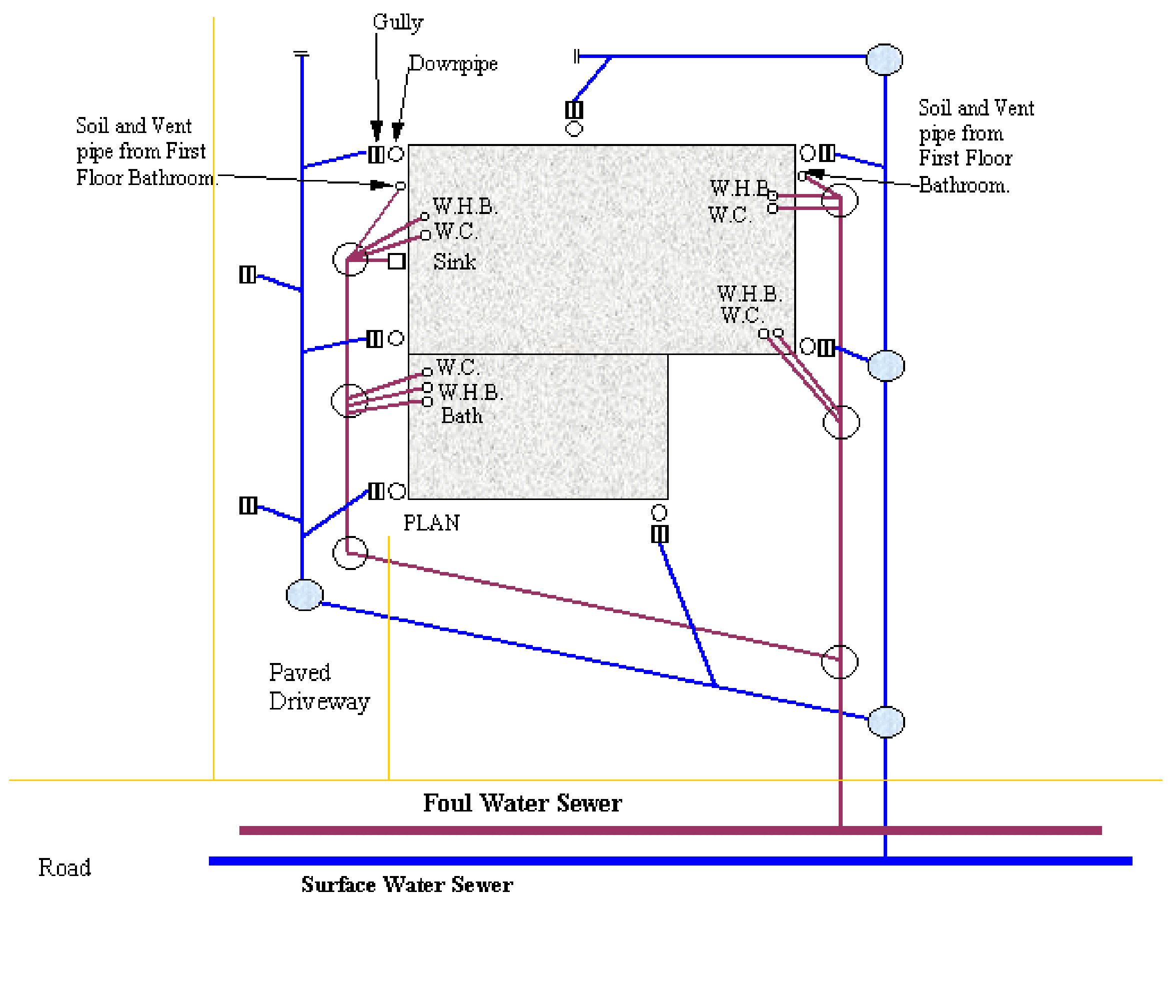
- Hand-pour flushed type 20 mm




**Basic principles of good drainage system**

1.) Material have adequate strength and durability.
2.) Diameter of drain to be as small as possible.
3.) Accessibility- every part of drain within reach for inspection and maintenance.
4.) Laid in straight runs as far as possible.
5.) Drains laid to a gradient.
6.) Inlet be trapped. To prevent entry of foul air into building.
7.) Access fittings - Inspection chamber, rodding eye, manhole - placed at changes of direction and gradient.
8.) Inspection chambers placed at junctions.
9.) Junctions between drains arranged so that incoming drain joints at oblique angle in direction of flow.
10.) Avoid drain under buildings if possible.

Typical layout plan showing house drainage connections




DRAINAGF LAYOUT

## Maintenance of house drainage

For efficient working of the house drainage system it should be properly maintained and cleaned at regular intervals. Following points should be carefully looked at:

1. Entry of undesired elements - should take extreme precautions to avoid entry of undesired elements in the system such as grit, sand, decayed fruits, pieces of cloths, leaves, etc.
2. Flushing - advisable to flush the system once or twice in a day in order to maintain it in proper working order.
3. Inspection - various unit 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

5. Use of disinfectants - Disinfectants should be freely used in the lavatory blocks, bathrooms, etc., to maintain good sanitary conditions in the building.
6. Workmanship - Laying of drains and fixing of pipes should be carried out by licensed or authorised plumbers only.

Sewage Pumping

Necessity of sewage pumping-

Although sewage flows in the sewers under gravity only, but still there sre occasions when it may have to be lifted from lower level to higher level. Pumping becomes essential under following situations,

1) Sewage of lavatory blocks located on basements of houses has to be lifted, because the level of basement is lower than the invert level of municipal sewer or street sewer.

2) Sewage may have to be lifted in flat areas at certain intervals to avoid costly excavation and to lay sewer at reasonable depths below the ground level. Reason for not taking sewers deep may be sub soil water also.

3) To lift the sewage of the flow laying areas of the city and put it into street sewers at higher level.

4) To lift the sewage from outfall sewers to treatment plant or to natural rivers or streams, if outfall sewer lies at lower level than the level of the disposal system.

5) At treatment also, when it is required to make the sewage flow under gravity, if it is a low level.

Some special problems in Sewage Pumping.

1) Sewage has lot of suspended, floating and solid matters. They cause very frequent problem of clogging of pumps.

2) Sewage contains organic and inorganic wastes. They may act upon the pumping equipment and may corrode then to reduce their life.

3) The flow of incoming sewage is not constant but keeps on fluctuating from time to time. Due to this aspect pumping has to be adjusted accordingly by operating different units of pumps having different pumping capacities.

4) Sewage pump has to be very reliable, otherwise it will cause flooding and lot of nuisance. Since sewage also carries a number of disease producing bacteria, it may prove even worse for public health if flooding nuisance is allowed to prevail. Therefore reliability of pumps has to be of very importance.

Preparation of Sewage for Pumping.

It is desirable to remove as much of the coarser floating matter as possible, before sewage is pumped. Hence, screens are constructed used in advance of the pumps to reduce the possibility of a pump becoming clogged and damaged by large sticks and rags. Grit is also removed in advance of pumping, where possible. This measure reduces wear on pumps. A grit chamber and a screening device are commonly included among units of a sewage treatment plant.

Requirements of Sewage Pumps.

The following are the requirements of a good sewage pump;

(1) It should be efficient enough to lift the sewage for higher heads.
(2) It should be reliable
(3) It should be cheap in initial cost and maintenance.
(4) It would not be corroded by organic and inorganic wastes.
(5) It should be of non clog type, so that it will not clog due to the suspended solids and floating solids in the sewage.
(6) It should require less space for installation.
(7) It should not make more noise during work.
(8) As the quantity of sewage is continuously varying, the pumps must allow fluctuations in it and can be easily adjusted.

Centrifugal Pumps.

Sewage contains rags, sticks, etc and such other materials which render the sewage difficult to pump. Therefore

(1) Preliminary screening and grit chambers.
(2) Sump or wet well.
(3) Pump room or dry well.
(4) Pumps with driving motor or engine.
(5) Miscellaneous accessories such as pipes, valves, fittings, valves, flow recorder, emergency overflows etc.

(1) Preliminary screening and grit channels: Since the sewage reaching the pump station contains large amount of sand, gravel, rags, paper, leaves etc, before sewage is pumped, it is most essential to remove all these things as far as possible to prevent wear and tear of pumping machinery and increasing its life. Large floating matters are removed from sewage by passing it through screens.
(2) Wet well: The sewage received from the city at a pumping station is collected in a tank known as sump or wet well. The capacity of the wet well is such that it can store D.W.F

of at least 2 hours, which is the maximum period during which pumps can be repaired or replaced.

(3) Pump room: This is also called as dry-well. It is placed in a convenient place and pumps are installed inside it. Its location should be such, so that pumps can easily function.

Maintenance of Sewers.

Maintenance of sewers consists mainly of the removal or prevention of stoppages, cleaning of sewers and other sewer appurtenances and repair works. Maintenance of sewers become costly when they are laid on flat gradients and tree roots find easy entrance in sewers through defective joints. The maximum expenditure in maintenance courses on the cleaning of sewers which may have been clogged due to deposition of silt, grease, and oily materials.

For good maintenance up-to-date plants of sewer system showing location of manholes, and other appurtenances, direction of flow, house sewers, grades of sewer lines etc are necessary. Before actual cleaning and repair work the inspection is done.

During inspection, the clogging of sewers, breakage of pipes etc, are noted and then the cleaning and repairs works are done later on.

Causes of Damage to Sewers

Following are the main causes of damage to the sewers;

(1) Bad workmanship and use of low specification materials.
(2) Faulty design.
(3) Excessive superimposed load.
(4) Settlement due to loose foundation or made up ground.
(5) Deterioration of sewers due to corrosion.
(6) Old age of the sewer.

# Unit-IV

## SAMPLING:

### Collection and Preservation of Samples

The objective of sampling is to collect representative sample. Representative sample by means a sample in which relative proportions or concentration of all pertinent components will be the same as in the material being sampled. Moreover, the same sample will be handled in such a way that no significant changes in composition occur before the tests are made. The sample volume shall optimal small enough that it can be transported and large enough for analytical purposes.
Because of the increasing placed on verifying the accuracy and representatives of data, greater emphasis is placed on proper sample collection, tracking, and preservation techniques. Often laboratory personnel help in planning a sampling program, in consultation with the user of the test results. Such consultation is essential to ensure selecting samples and analytical methods that provide a sound and valid basis for answering the questions that prompted the sampling and that will meet regulatory and/or project-specific requirements.

General Requirements

Obtain a sample that meets the requirements of the sampling program and handle it so that it does not deteriorate or become contaminated before it is analyzed.

Ensure that all sampling equipment is clean and quality-assured before use. Use sample containers that are clean and free of contaminants. Bake at 450°C all bottles to be used for organic analysis sampling.

Fill sample containers without pre-rinsing with sample; pre-rinsing results in loss of any pre-added preservative and sometimes can bias results high when certain components adhere to the sides of the container. Depending on determinations to be performed, fill the container full (most organic compound determinations) or leave space for aeration, mixing, etc. (microbiological and inorganic analyses). If the bottle already contains preservative, take care not to overfill the bottle, as preservative may be lost or diluted. Except when sampling for analysis of volatile organic compounds, leave an air space equivalent to approximately 1% of the container volume to allow for thermal expansion during shipment.

Special precautions are necessary for samples containing organic compounds and trace metals. Since many constituents may be present at low concentrations (micro-grams or nanograms per liter), they may be totally or partially lost or easily contaminated when proper sampling and preservation procedures are not followed.
Composite samples can be obtained by collecting over a period of time, or at many different over a period of time, depth, or at many different sampling points. The details of collection vary with local conditions, so specific recommendations are not universally applicable. Sometimes it is more informative to analyze numerous separate samples instead of one composite so that variability, maxima, and minima can be determined.

Because of the inherent instability of certain properties and compounds, composite sampling for some analytes is not recommended where quantitative values are desired (examples in-residual, iodine, hexavalent chromium, nitrate, volatile organic compounds, radon-222, dissolved oxygen, ozone, temperature, and pH). In certain cases, such as for BOD, composite samples are routinely by regulatory agencies. Refrigerate composite samples for BOD and nitrite.

Important factors affecting results are the presence of suspended matter or turbidity, the method chosen for removing a sample from its container, and the physical and chemical brought about by storage or aeration. Detailed procedures are essential when processing (blending, sieving, filtering) samples to be analyzed for trace constituents, especially metals and organic compounds. Some determinations can be invalidated by contamination during processing. Treat each sample individually with regard to the substances to be determined, the amount and nature of turbidity present, and other conditions that may influence the results.

For metals it often is appropriate to collect both a filtered and an unfiltered sample to differentiate between total and dissolved metals present in the matrix. Be aware that some metals may partially sorb to filters. Beforehand, determine the acid requirements to bring the pH to $<2$ on a separate sample. Add the same relative amount of acid to all samples; use ultrapure acid preservative to prevent contamination. When filtered samples are to be collected, filter them, if possible, in the field, or at the point of collection before preservation with acid. Filter samples in a laboratory-controlled environment if field conditions could cause error or contamination; in this case filter as soon as possible. Often slight turbidity can be tolerated if experience shows that it will cause no interference in gravimetric or volumetric tests and that its influence can be corrected in colorimetric tests, where it has potentially the greatest interfering effect. Sample collector must state whether or not the sample has been filtered.

Record of sample shall be as follows:

General information

- Sample identification number
- Location
- Sample collector
- Date and hour
- Sample type (Grab or composite)

Specific information

- Water temperature
- Weather
- Stream flow
- Water level
- Any other information

**Sampling Techniques**

Two types of sampling techniques are used: grab and composite.

a. *Grab samples:* Grab samples are single collected at a specific spot at a site over a short period of time (typically seconds or minutes). Thus, they represent a ―snapshot" in both space and time of a sampling area. Discrete grab samples are taken at a selected location, depth, and time. Depth-integrated grab samples are collected over a predetermined part of the entire depth of a water column, at a selected location and time in a given body of water.

Grab samples consist of either a single discrete sample or individual samples collected over a period of time not to exceed 15 minutes. The grab sample should be representative of the wastewater conditions at the time of sample collection. The sample volume depends on the type and number of analyses to be performed.

A sample can represent only the composition of its source at the time and place of collection. However, when a source is known to be relatively constant in composition over an extended time or over substantial distances in all directions, then the sample may represent a longer time period and/or a larger volume than the specific time and place at which it was collected. In such circumstances, a source may be represented adequately by single grab samples. Examples are protected groundwater supplies, water supplies receiving conventional treatment, some well-mixed surface waters, but rarely, wastewater streams, rivers, large lakes, shorelines, estuaries, and groundwater plumes.

When a source is known to vary with time, grab samples collected at suitable intervals and analyzed separately can document the extent, frequency, and duration of these variations. Choose sampling intervals on the basis of the expected frequency of changes, which may vary from as little as 5 min to as long as 1h or more. Seasonal variations in natural systems may necessitate sampling over months. When the source composition varies in space (i.e. from location to location) rather than time, collect samples from appropriate locations that will meet the objectives of the study (for example, upstream and downstream from a point source, etc.).

b. *Composite samples:* Composite samples should provide a more representative sampling of heterogeneous matrices in which the concentration of the analytes of interest may vary over short periods of time and/or space. Composite samples can be obtained by combining portions of multiple grab samples or by using specially designed automatic sampling devices. Sequential (time) composite samples are collected by using continuous, constant sample pumping or by mixing equal water volumes collected at regular time intervals. Flow-proportional composites are collected by continuous pumping at a rate proportional to the flow, by mixing equal volumes of water collected at time intervals that are inversely proportional to the volume of flow, or by mixing volumes of water proportional to the flow collected during or at regular time intervals.

Advantages of composite samples include reduced costs of analyzing a large number of samples, more representative samples of heterogeneous matrices, and larger sample sizes when amounts of test samples are limited. Disadvantages of composite samples include loss of analyte relationships in individual samples, potential dilution of analytes below detection levels, increased potential analytical interferences, and increased possibility of analyte interactions. In addition, use of composite samples may reduce the number of samples analyzed below the required statistical need for specified data quality objectives or project-specific objectives.

Do not use composite samples with components or characteristics subject to significant and unavoidable changes during storage. Analyze individual samples as soon as possible after collection and preferably at the sampling point. Examples are dissolved gases, residual chlorine, soluble sulfide, temperature, and pH. Changes in components such as dissolved oxygen or carbon dioxide, pH, or temperature may produce secondary changes in certain inorganic constituents such as iron, manganese, alkalinity, or hardness. Some organic analytes also may be changed by changes in the foregoing components. Use time-composite samples only for determining components that can be demonstrated to remain unchanged under the conditions of sample collection, preservation, and storage.

Collect individual portions in a wide-mouth bottle every hour (in some cases every half hour or even every 5 min) and mix at the end of the sampling period or combine in a single bottle as collected. If preservatives are used, add them to the sample bottle initially so that all portions of the composite are preserved as soon as collected.

Automatic sampling devices are available; however, do not use them unless the sample is preserved as described below. Composite samplers running for extended periods (week to months) should undergo routine cleaning of containers and sample lines to minimize sample growth and deposits.

Composite samples are collected over time, either by continuous sampling or by mixing discrete samples. A composite sample represents the average wastewater characteristics during the compositing period. Various methods for compositing are available and are based on either time or flow proportioning. The choice of a flow proportional or time composite sampling scheme depends on the permit requirements, variability of the wastewater flow or concentration of pollutants, equipment availability and sampling location. The investigator must know each of these criteria before a sampling program can be initiated. Generally, a time composite is acceptable. However, in enforcement cases where strict adherence to permit requirements are necessary, a flow proportional sample is preferable, if possible.

A time composite sample consists of equal volume discrete sample aliquots collected at constant time intervals into one container. A time composite sample can be collected either manually or with an automatic sampler.

A flow proportional composite sample can be collected using one of two methods. One method consists of collecting a constant sample volume at varying time intervals proportional to the wastewater flow. For the other method, the sample is collected by varying the volume of each individual aliquot proportional to the flow, while maintaining a constant time interval between the aliquots.

Flow proportional samples can be collected directly with an automatic sampler that is connected to a compatible flow measuring device. An automatic sampler can also be used to collect discrete samples.

c. *Integrated (discharge*-weighted) samples: For certain purposes, the information needed is best provided by analyzing mixtures of grab samples collected from different points simultaneously, or as nearly so as possible, using discharge-weighted methods such as equal-width increment (EWI) or equal discharge-increment (EDI) procedures and equipment. An example of the need for integrated sampling occurs in a river or stream that varies in composition across its width and depth. To evaluate average composition or total loading, use a mixture of samples representing various points in the cross-section, in proportion to their relative flows. The need for integrated samples also may exist if combined treatment is proposed for several separate wastewater streams, the interaction of which may have a significant effect on treatability or even on composition. Mathematical prediction of the interactions among chemical components may be inaccurate or impossible and testing a suitable integrated sample may provide useful information.

Both lakes and reservoirs show spatial variations of composition (depth and horizontal location). However, there are conditions under which neither total nor average results are especially useful, but local variations are more important. In such cases, examine samples separately (i.e., do not integrate them).
Preparation of integrated samples usually requires equipment designed to collect a sample water uniformly across the depth profile. Knowledge of the volume, movement, and composition of the various parts of the water being sampled usually is required. Collecting integrated samples is a complicated and specialized process that must be described in a sampling plan.

Sampling Methods

a. *Manual sampling:* Manual sampling involves minimal equipment but may be unduly costly and time-consuming for routine or large-scale sampling programs. It requires trained field technicians and is often necessary for regulatory and research investigations for which critical appraisal of field conditions and complex sample collection techniques are essential. Manually collect certain samples, such as waters containing oil and grease.
b. *Automatic sampling:* Automatic samplers can eliminate human errors in manual sampling, can reduce labor costs, may provide the means for more frequent sampling, and are used increasingly. Be sure that the automatic sampler does not contaminate the sample. For example, plastic components may be incompatible with certain organic compounds that are soluble in the plastic parts or that can contaminated (e.g., from phthalate esters) by contact with them. If sample constituents are generally known, contact the manufacturer of an automatic sampler regarding potential incompatibility of plastic components.

Program an automatic sampler in accordance with sampling needs. Carefully match pump speeds and tubing sizes to the type of sample to be taken.
c. *Sorbent sampling:* Use of solid sorbents, particularly membrane-type disks, is becoming more frequent. These methods offer advantages of rapid, inexpensive sampling if the analytes of interest can be adsorbed and desorbed efficiently and the water matrix is free of particulates that plug the sorbent.

Sample Containers

The type of sample containers used is of utmost importance. Test sample containers and document that they are free of analytes of interest, especially when sampling and analyzing for very low analyte levels. Containers typically are made of plastic or glass, but one material may be preferred over the other. For example, silica, sodium, and boron may be leached from soft glass but not plastic, and trace levels of same pesticides and metals may sorb onto the walls of glass containers. Thus, hard glass containers are preferred. For samples containing organic compounds, do not use plastic containers except those made of fluorinated polymers such as polytetrafluoroethylene (PTFE).

Some sample analytes may dissolve (be absorbed) into the walls of plastic containers; similarly, contaminants from plastic containers may leach into samples. Avoid plastics wherever possible because of potential contamination from phthalate esters. Containers failure due to breakdown of the plastic is possible. Therefore, use glass containers for all organics analyses such as volatile organics, semivolatile organics, pesticides, PCBs, oil and grease. Some analytes (e.g., bromine-containing compounds and some pesticides. Polynuclear aromatic compounds, etc.) are light sensitive; collect them in amber-colored glass containers to minimize photodegradation. Container caps, typically plastic, also can be a problem. Do not use caps with paper liners. Use foil or PTFE liners but be aware that metal liners can contaminate samples collected for metals analysis and they may also react with the sample if it is acidic or alkaline. Serum vials with PTFE-lined rubber or plastic septa are useful.

In rare situations it may be necessary to use sample containers not specifically prepared for use, or otherwise unsuitable for the particular situation; thoroughly document these deviations. Documentations should include type and source of container, and the preparation technique, e.g., acid washed with reagent water rinse. For QA purposes the inclusion of a bottle blank may be necessary.

**Special Precautions for Wastewater Sampling**

- A clean pair of new, non-powdered, disposable gloves will be worn each time a different location is sampled and the gloves should be donned immediately prior to sampling. The gloves should not come in contact with the media being sampled and should be changed any time during sample collection when their cleanliness is compromised.

- Sample containers for samples suspected of containing high concentrations of contaminants shall be stored separately.

- Sample collection activities shall proceed progressively from the least suspected contaminated area to the most suspected contaminated area. Samples of waste or highly contaminated media must not be placed in the same ice chest as environmental (i.e., containing low contaminant levels) or background/control samples.

- If possible, one member of the field sampling team should take all the notes and photographs, fill out tags, etc., while the other members collect the samples.

- Field investigators must use new, verified certified-clean disposable or non-disposable equipment should be cleaned according to procedures.

**Sample Handling and Preservation Requirements**

1. Wastewater samples will typically be collected either by directly filling the sample container or by using an automatic sampler or other device.
2. During sample collection, if transferring the sample from a collection device, make sure that the device does not come in contact with the sample containers.
3. Place the sample into appropriate, labeled containers. Samples collected for VOC analysis must not have any headspace. All other sample containers must be filled with an allowance for spillage.

4. All samples requiring preservation must be preserved as soon as practically possible, ideally immediately at the time of sample collection. If preserved VOC vials are used, these will be preserved with concentrated hydrochloric acid prior to departure for the field investigation. For all other chemical preservatives, use the appropriate chemical preservative generally stored in an individual single-use vial. The adequacy of sample preservation will be checked after the addition of the preservative for all samples, except for the samples collected for VOC analysis. If it is determined that a sample is not adequately preserved, additional preservative should be added to achieve adequate preservation.
5. All samples preserved using a pH adjustment (except VOCs) must be checked, using pH strips, to ensure that they were adequately preserved. This is done by pouring a small volume of sample over the strip. Do not place the strip in the sample. Samples requiring reduced temperature storage should be placed on ice immediately.

## WASTE WATER CHARACTERSTICS

**Wastewater** is simply that part of the water supply to the community or to the industry which has been used for different purposes and has been mixed with solids either suspended or dissolved. Wastewater is 99.9% water and 0.1% solids. The main task in treating the wastewater is simply to remove most or all of this 0.1% of solids.

**Type of wastewater from household**

| Type of Wastewater | Source of wastewater |
|---|---|
| Gray water | Washing water from the kitchen, bathroom, laundry (without faeces and urine) |
| Black water | Water from flush toilet (faeces and urine with flush water) |
| Yellow water | Urine from separated toilets and urinals |
| Brown water | Black water without urine or yellow water |

**Why do we need to treat waste water ?**

•To prevent groundwater pollution
•To prevent sea shore
•To prevent soil
•To prevent marine life
•Protection of public health
•To reuse the treated effluent
For agriculture
For groundwater recharge
For industrial recycle
•Solving social problems caused by the accumulation of wastewater

•**Protecting the public health:** Wastewater contains pathogenic microorganisms lead to dangerous diseases to humans and animals. Hazardous matter such as heavy metals that are toxic Produces odorous gases and bad smell

•**Protecting the environment:** Raw Wastewater leads to septic conditions in the environment and consequently leads to the deterioration of surface and groundwater quality and pollutes the soil. Raw wastewater is rich with nitrogen and phosphorus (N, P) and leads to the phenomena of EUTROPHICATION. EUTROPHICATION is the growth of huge amounts of algae and other aquatic plants leading to the deterioration of the water quality. Raw wastewater is rich with organic matter which consumes oxygen in aquatic environment. Raw wastewater may contains toxic gases and volatile organic matter

**Physical, chemicals and biological characteristics /properties of wastewater**

| **Characteristic** | **Sources** |
|---|---|
| Physical properties: Color | Domestic and industrial wastes, natural decay of organic materials |
| Odor | Decomposing wastewater, industrial wastes. |
| Solids | Domestic water supply, domestic and industrial wastes, soil erosion, inflow infiltration |
| Temperature | Domestic and industrial wastes |
| Chemical constituents: Organic: Carbohydrates | Domestic, commercial, and industrial wastes |
| Fats, oils, and grease | Domestic, commercial, and industrial wastes |
| Pesticides | Agricultural wastes |
| Phenols | Industrial wastes |
| Proteins | Domestic, commercial, and industrial wastes |
| Priority pollutants | Domestic, commercial, and industrial wastes |
| Surfactants | Domestic, commercial, and industrial wastes |
| Volatile organic compounds | Domestic, commercial, and industrial wastes |
| Other | Natural decay of organic materials |
| Inorganic: Aikalinity | Domestic wastes, domestic water supply, groundwater infiltration |
| Chlorides | Domestic wastes, domestic water supply, groundwater infiltration |
| Heavy metals | Industrial wastes |
| Nitrogen | Domestic and agricultural wastes |
| PH | Domestic, commercial, and industrial wastes |
| Phosphorus | Domestic, commercial, and industrial wastes natural runoff |
| Priority polluter Sulfur | Domestic water supply; doestic, commercial. And industrial wastes |

| | |
|---|---|
| Gases: Hydrogen sulfide | Decomposition of domestic wastes |
| Methane | Decomposition of domestic wastes |
| Oxygen | Domestic water supply , surface-water infiltration |
| Biological constituents: Animals | Open watercourses and treatment plants |
| Plants | Open watercourses and treatment plants |
| Eubacteria | Domestic wastes, surface water infiltration, treatment plants . |
| Archaebacteria | Domestic wastes, surface-water infiltration, treatment plants |
| Viruses | Domestic wastes |

**Physical characteristics-Solids**

• Solids are classified into three main types:

1.Total Solids (TS): All the matter that remains as residue upon evaporation at 103oC to 105oC.

2. Settleable solids: Settleable solids are measured as ml/L, which is an approximate measure of the sludge that can be removed by primary sedimentation.

3. Suspended solids (SS) and Filterable solids (FS).

**Physical characteristics-Odor**

Odor is produced by gas production due to the decomposition of organic matter or by substances added to the wastewater.

Detection of odour: Odour is measured by special instruments such as the Portable $H_2S$ meter which is used for measuring the concentration of hydrogen sulfide.

| Compound | Chemical Formula | Odor quality |
|---|---|---|
| Amines | CH3NH2, (CH3) 3H | Fishy |
| Ammonia | NH3 | Ammoniacal |
| Diamines | NH2(CH2)4NH2, (CH2)5NH2H25 | Rotten eggs |
| Mercaptans (E. g, methyl and ethyl) | CH3SH, CH3(CH2) SH | Decayed cabbage |
| Organic sulfides | | Rotten cabbage |
| Skatole | | Fecal matter |

**Physical characteristics-Temperature**

Temperature of wastewater is commonly higher than that of water supply. Depending on the geographic location the mean annual temperature varies in the range of 10 to 21$^{o}$C with an average of 16$^{o}$C.

Importance of temperature:-

Affects chemical reactions during the wastewater treatment process.
Affects aquatic life.
Oxygen solubility is less in worm water than cold water. Optimum temperature for bacterial activity is in the range of 25°C to $35^0$C. Aerobic digestion and nitrification stop when the temperature rises to $50^o$C. When the temperature drops to about 15°c, methane producing bacteria become in active. Nitrifying bacteria stop activity at about 5°c.

**Color:-**
Fresh waste water- light brownish gray.
With time -dark gray
More time- black (septic).
Sometimes - pink due to algae or due to industrial colors.

**Turbidity:-**
It's a measure of the light –transmitting properties of water.

**Chemical characteristics of wastewater:-**
Points of concern regarding the chemical characteristics of wastewater are:
-Organic matter -Measurements of organic matter
-Inorganic matter
-Gases
-pH
**Organic matter (CaHbOc).**
75% -SS organic. (Suspended Solids)
40% -FS organic. (Filtered Solids)
Organic mater is derived from animals & plants and man activities.
Proteins -(40-60%).
Carbohydrates- (25-50%).
Fats, Oils, and Grease- (10%).

**DISSOLVED OXYGEN**

Dissolved oxygen analysis measures the amount of gaseous oxygen ($O_2$) dissolved in an aqueous solution. Dissolved oxygen is one of the most important parameters in aquatic systems. This gas is an absolute requirement for the metabolism of aerobic organisms and also influences inorganic chemical reactions. Therefore, knowledge of the solubility and dynamics of oxygen distribution is essential to interpreting both biological and chemical processes within water bodies. Oxygen gets into water by diffusion from the surrounding air, by aeration (rapid movement) and as a waste product of photosynthesis. The amount of dissolved oxygen gas is highly dependent on temperature. Atmospheric pressure also has an effect on dissolved oxygen. The amount of oxygen (or any gas) that can dissolve in pure water (saturation point) is inversely proportional to the temperature of water. The warmer the water, the less dissolved oxygen.

**Environment Significance**

In a nutrient-rich water body the dissolved oxygen is quite high in the surface water due to increased photosynthesis by the large quantities of algae. However, dissolved oxygen tends to be depleted in deeper waters because photosynthesis is reduced due to poor light penetration and due to the fact that dead phytoplankton (algae) falls toward the bottom using up the oxygen as it decomposes. In a nutrient-poor water body there is usually less difference in dissolved oxygen from surface to bottom. This difference between surface and bottom waters is exaggerated in the summer in reservoirs, stream-pools, and embayment when thermal layering occurs which prevents mixing. The surface may become supersaturated with oxygen (>100%) and the bottom anoxic (virtually no oxygen). Shallower reservoirs and actively flowing shallow streams generally are kept mixed due to wind action in the shallow reservoirs and physical turbulence created by rocks in the stream beds.

Adequate dissolved oxygen is needed and necessary for good water quality. Oxygen is a necessary element to all forms of life. Adequate oxygen levels are necessary to provide for aerobic life forms which carry on natural stream purification processes. As dissolved oxygen levels in water drop below 5.0 mg/L, aquatic life is put under stress. The lower the concentration, the greater the stress. Oxygen levels that remain below 1-2 mg/L for a few hours can result in large fish kills. Total dissolved oxygen concentrations in water should not exceed 110 percent. Concentrations above this level can be harmful to aquatic life. Fish in waters containing excessive dissolved gases may suffer from "gas bubble disease"; however, this is a very rare occurrence. The bubbles or emboli block the flow of blood through blood vessels causing death. Aquatic invertebrates are also affected by gas bubble disease but at levels higher than those lethal to fish.

A high DO level in a community water supply is good because it makes drinking water taste better. However, high DO levels speed up corrosion in water pipes. For this reason, industries use water with the least possible amount of dissolved oxygen. Water used in very low pressure boilers have no more than 2.0 ppm of DO, but most boiler plant operators try to keep oxygen levels to 0.007 ppm or less. Dissolved oxygen (DO) refers to the amount of oxygen dissolved in water and is particularly important in limnology (aquatic ecology). Oxygen comprises approximately 21% of the total gas in the atmosphere; however, it is much less available in water. The amount of oxygen water can hold depends upon temperature (more oxygen can be dissolved in colder water), pressure (more oxygen can be dissolved in water at greater pressure), and salinity (more oxygen can be dissolved in water of lower salinity). Many lakes and ponds have anoxic (oxygen deficient) bottom layers in the summer because of decomposition processes depleting the oxygen.

The amount of dissolved oxygen often determines the number and types of organisms living in that body of water. For example, fish like trout are sensitive to low DO levels (less than eight parts per million) and cannot survive in warm, slow-moving streams or rivers. Decay of organic material in water caused by either chemical processes or microbial action on untreated sewage or dead vegetation can severely reduce dissolved oxygen concentration. This is the most common cause of fish kills, especially in summer months when warm water holds less oxygen anyway.

Dissolved oxygen (DO) refers to the volume of oxygen that is contained in water. Oxygen enters the water as rooted aquatic plants and algae undergo photosynthesis, and as oxygen is transferred across the air-water interface. The amount of oxygen that can be held by the water depends on the water temperature, salinity, and pressure. Gas solubility increases with decreasing temperature (colder water holds more oxygen). Gas solubility increases with decreasing salinity (freshwater holds more oxygen than does saltwater). Both the partial pressure and the degree of saturation of oxygen will change with altitude. Finally, gas solubility decreases as pressure decreases.

Thus, the sun-warmed water will remain at the surface of the water body (forming the epilimnion), while the more dense, cooler water sinks to the bottom (hypolimnion). The layer of rapid temperature change separating the two layers is called the thermocline. At the beginning of the summer, the hypolimnion of the lake will contain more dissolved oxygen because colder water holds more oxygen than warmer water. However, as time progresses, an increased number of dead organisms from the epilimnion sink to the bottom and are broken down by microorganisms. Continued microbial decomposition eventually results in an oxygen-deficient hypolimnion. If the lake has high concentrations of nutrients, this process may be accelerated. When the growth rate of microorganisms is not limited by a specific nutrient, such as phosphorus, the dissolved oxygen in the lake could be depleted before the summer's end.

Microbes play a key role in the loss of oxygen from surface waters. Microbes use oxygen as energy to break down long-chained organic molecules into simpler, more stable end products such as carbon dioxide, water, phosphate and nitrate. As microbes break down the organic molecules, oxygen is removed from the system and must be replaced by exchange at the air-water interface. Each step above results in consumption of dissolved oxygen. If high levels of organic matter are present in a water, microbes may use all available oxygen. This does not mean, however, that the removal of microbes from the ecosystem would solve this problem. Although microbes are responsible for decreasing levels of dissolved oxygen, they play a very important role in the aquatic ecosystem. If dead matter is not broken down it will "pile up," much as leaves would if they were not broken down each year.

**Dissolved oxygen and its importance**

The stream system both produces and consumes oxygen. It gains oxygen from the atmosphere and from plants as a result of photosynthesis. Running water, because of its churning, dissolves more oxygen than still water, such as that in a reservoir behind a dam. Respiration by aquatic animals, decomposition, and various chemical reactions consume oxygen.

Wastewater from sewage treatment plants often contains organic materials that are decomposed by microorganisms, which use oxygen in the process. (The amount of oxygen consumed by these organisms in breaking down the waste is known as the biochemical oxygen demand or BOD. A discussion of BOD and how to monitor it is included at the end of this section.) Other sources of oxygen-consuming waste include storm water runoff from farmland or urban streets, feedlots, and failing septic systems.

Oxygen is measured in its dissolved form as dissolved oxygen (DO). If more oxygen is consumed than is produced, dissolved oxygen levels decline and some sensitive animals may move away, weaken, or die.

DO levels fluctuate seasonally and over a 24-hour period. They vary with water temperature and altitude. Cold water holds more oxygen than warm water and water holds less oxygen at higher altitudes. Thermal discharges, such as water used to cool machinery in a manufacturing plant or a power plant, raise the temperature of water and lower its oxygen content. Aquatic animals are most vulnerable to lowered DO levels in the early morning on hot summer days when stream flows are low, water temperatures are high, and aquatic plants have not been producing oxygen since sunset.

**Measurements of organic matter:-**

Many parameters have been used to measure the concentration of organic matter in wastewater.

The following are the most common used methods:

**Biological Oxygen Demand (BOD):**

**Biochemical oxygen demand** or **B.O.D** is the amount of dissolved oxygen needed by aerobic biological organisms in a body of water to break down organic material present in a given water sample at certain temperature over a specific time period. The term also refers to a chemical procedure for determining this amount. This is not a precise quantitative test, although it is widely used as an indication of the organic quality of water. The BOD value is most commonly expressed in milligrams of oxygen consumed per litre of sample during 5 days of incubation at 20 °C and is often used as a robust surrogate of the degree of organic pollution of water.

$BOD_5$ is the oxygen equivalent of organic matter. It is determined by measuring the dissolved oxygen used by microorganisms during the biochemical oxidation of organic matter in 5 days at $20^oC$.

Measurement of amount of oxygen used by bacteria to metabolize organic material in water

**Organic material + bacteria + $O_2 \rightarrow CO_2 + H_2O$**

**Five-day BOD ($BOD_5$)** is performed at 20°C in dark over five days, with added seed (bacteria), nutrients and oxygen. Expressed as mg/L.

**Uses of BOD Test**

Used as a measurement of waste strength

- is an effluent discharge limitation parameter in mill permits
- Used as basis for designing & evaluating biological treatment systems
- to determine amount of Oxygen needed to stabilize organic matter in waste
- to determine size of wastewater treatment facilities
- to measure efficiency of some treatment processes

**Limitations of $BOD_5$**

- Test period is too long not good for process control

- Only 60-70% of soluble organic matter is metabolized in five days

• Inorganic oxidation may also occur:

1. Inorganic oxidation of sulfur or iron:
   $\mathbf{S^{--} + 2\,O_2 \rightarrow SO_4^{--}}$ or $\mathbf{4Fe^{++} + 3O_2 \rightarrow 2Fe_2O_3}$
2. Nitrification may also occur if ammonia is present Conversion of ammonia to nitrate:
   $\mathbf{NH_3 + 2\,O_2 \rightarrow HNO_3 + H_2O}$

**Details of Nitrification**

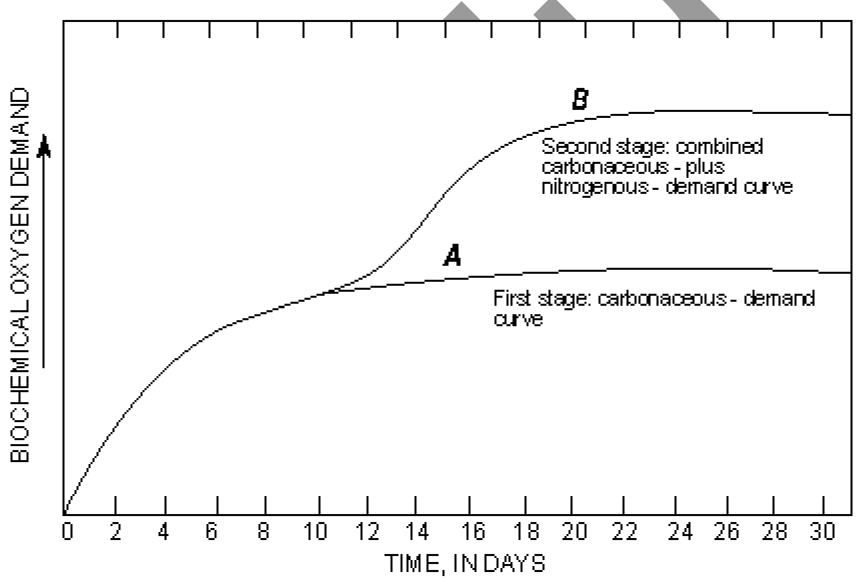
Nitrifying bacteria can oxidize ammonia, common to many waste streams, resulting in nitrogenous oxygen demand (NBOD).

Conversion of ammonia to nitrite
*(Nitrosomonas bacterium)*:

$NH_3 + 3/2\ O_2 \rightarrow HNO_2 + H_2O$

Conversion of nitrite to nitrate
*(Nitrobacter bacterium)*:
$HNO_2 + 1/2\ O_2 \rightarrow HNO_3$

Nitrifying bacteria can be suppressed by chemical treatment, pasteurization, or chlorination followed by de-chlorination.




Nitrification usually begins 6 to 10 days after the start of the BOD incubation period.

There are two stages of decomposition in the BOD test: a carbonaceous stage and a nitrogenous stage.

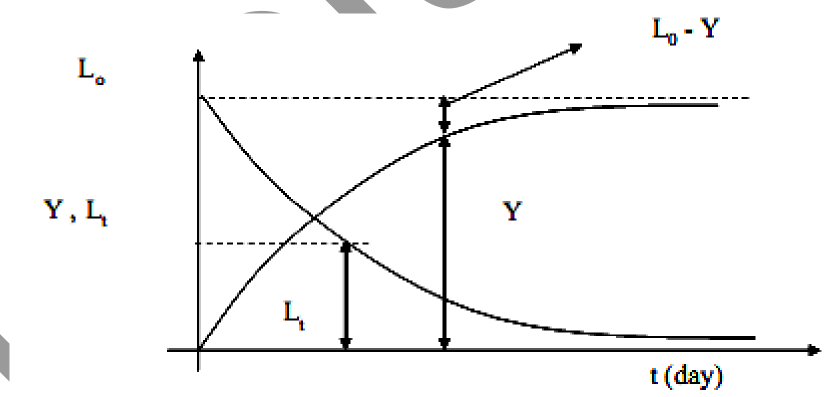
**The carbonaceous stage**, or first stage, represents that portion of oxygen demand involved in the conversion of organic carbon to carbon dioxide.

**The nitrogenous stage**, or second stage, represents a combined carbonaceous plus nitrogeneous demand, when organic nitrogen, ammonia, and nitrite are converted to nitrate. Nitrogenous oxygen demand generally begins after about 6 days. For some sewage, especially discharge from wastewater treatment plants utilizing biological treatment processes, nitrification can occur in less than 5 days if ammonia, nitrite, and nitrifying bacteria are present. In this case, a chemical compound that prevents nitrification should be added to the sample if the intent is to measure only the carbonaceous demand. The results are reported as carbonaceous BOD (CBOD), or as $CBOD_5$ when a nitrification inhibitor is used.

The following are the theoretical equations used to calculate the BOD.
The Figure shown is used to describe the change of BOD with time. From the figure the following correlations are derived:
$L_0$ or (BOD ultimate) or UBOD.
Yt= BODt (BOD exerted).
$Lt = L_0e^{-kt}$ (BOD remain).
$BODt = L_0 - Lt = L_0 - L_0e^{-kt} = L_0 (1-e^{-kt})$
$BOD5 = L_0(1-e^{-k5})$
$K = 0.23d^{-1}$

Hoff-Arrhenius (_uh-REE-nee-us') relationship:
usually, $\mathbf{k_T = k_{20}\Theta^{T-20}}$,

$\Theta$= 1.047 or as given



**First stage BOD formulation.** At a given temperature, the rate at which BOD is satisfied at any time (*i.e.* rate of *deoxygenation*) may be assumed to be directly proportional to the amount of organic matter present in sewage. In other works, the exertion of BOD is considered to be first order reaction defined by

$$\frac{dL_t}{dt} = -K' . L_t$$

where $L_t$ = amount of first stage BOD remaining in the sample at any time $t$ (or oxygen equivalent of carbonaceous oxidisable organic matter present at any time $t$), expressed as mg/l.

$K'$ = rate constant signifying the rate of oxidation of organic matter, having a unit $(\text{day})^{-1}$. Its value depends upon the nature of organic matter present and the temperature during the reaction.

$t$ = time in days.

Integrating Eq. between time $t = 0$ (as which $L_t = L_0$, say) to $t = t$, we get

$$\int \frac{dL_t}{L_t} = -K' \int dt$$

or $$\log_e \frac{L_t}{L_0} = -K' t$$

or $$\frac{L_t}{L_0} = e^{-K't} = 10^{-Kt}$$

where the rate constant $K = \dfrac{K'}{2.303}$

In the above equation, $L_0$ is the oxygen equivalent of organic matter present in sewage at the beginning. Also $K$ is known as base 10 rate constant (or deoxigenation constant) while $K'$ is known base $e$ rate constant.

The amount of BOD remaining at any time $t$ is

$$L_t = L_0 (10^{-Kt})$$

Hence $y_t$, the amount of BOD that has been *exerted* at any time $t$ is given by :

$$y_t = (L_0 - L_t) = L_0 (1 - 10^{-Kt})$$

In the above equation, $y_t$ is the BOD of $t$ days (*i.e.* $y_t = \text{BOD}_t$).

5-day BOD is evidently given by

$$\text{BOD}_5 = y_5 = L_0 - L_5 = L_0 (1 - 10^{-5K})$$

**Problem:**
The BOD of a sewage incubated for one day at $30^0$C has been found to be 200 mg/l. What will be the 5-day $20^0$C BOD? Assume K = 0.12 per day at $20^0$C.

**Answer:**
**Data:**
1 day BOD at $30^0$ C, $Y_{1(30)} = 200$ mg/l
5 day BOD at $20^0$ C, $Y_{5(20)} =$?
Assume $K_{(20)} = 0.12$/d

**Calculations:**
Using Hoff – Arrhenius equation,
$K_{(T)} = K_{(20)}\ \theta^{(T - 20)}$
Assuming $\theta = 1.047$ for sewage,
$K_{(30)} = 0.12 \times 1.047^{(30 - 20)}$
$\mathbf{K_{(30)} = 0.19\ /d}$

Ultimate first stage BOD at $30^0$ is given by,
$Y_t = L\ (\ 1 - 10^{-k\,x\,t})$
$Y_{1(30)} = L_{(30)}\ (\ 1 - 10^{(-K(30)\,x\,t1)}\ )$

$$L_{(30)} = \frac{Y_{1(30)}}{(1 - 10^{-K(30)\,x\,t1}))} = \frac{200}{(1 - 10^{-(0.19\,x1)})}$$
$\mathbf{L_{(30)} = 564.42\ mg/l}$

Ultimate first stage BOD at $20^0$C is given by,
$L_{(T)} = L_{(20)}\ (\ 0.02\ T + 0.6\ )$
$L_{(30)} = L_{(20)}\ (\ 0.02\ T + 0.6\ )$
$564.42 = L_{(20)}\ (\ 0.02 \times 30 + 0.6\ )$
$\mathbf{L_{(20)} = 470.35\ mg/l}$

$Y_{5(20)} = L_{(20)}\ (\ 1 - 10^{(-K(20)\,x\,t5)}\ )$
$= 470.35(1-10^{-(0.12\,x\,5)})$
$\mathbf{= 352.2\ mg/l}$

**Problem:**
The BOD of a sewage sample incubated at $30^0$ C for 1 day has been found to be 100 mg/l. What will be the 5-day $20^0$ C BOD? Assume k = 0.12 at $20^0$C.

**Answer:**
$K_{(30)} = 0.19$/d
$L_{(30)} = 282.2$ mg/l
$L_{(20)} = 235.17$ mg/l
$Y_{5(30)} = 176.1$ mg/l

**Problem:**
The 5-day, $20^0$C BOD of a sample of sewage is 100 mg/L. Calculate its BOD at $37^0$ C, after 2 days.

**Answer:**
$K_{(37)} = 0.218/d$
$L_{(20)} = 146.24$ mg/l
$L_{(37)} = 195.96$ mg/l
$Y_{2(37)} = 124.15$ mg/l

**Problem:**
The BOD of a sewage incubated for 3 day at $25^0$C has been found to be 250 mg/l. What will be the 5-day $20^0$C BOD? Assume $K_1 = 0.1$ at $20^0$C.

**Answer:**

Reaction rate constant, $k_{(25)} = 0.126/d$
Ultimate BOD at $25^0$c, $L_{(25)} = 430.139$ mg/l
Ultimate BOD at $25^0$c, $L_{(20)} = 391.04$ mg/l
5-day $20^0$c BOD, $y_{5(20)} = 267.38$ mg/l

**Problem:**

The BOD of a sewage incubated for 3 day at $25^0$C has been found to be 205 mg/l. What will be the 5-day $20^0$C BOD? Assume $K_1 = 0.1$ at $20^0$C.

**Answer:**

$K_{(25)} = 0.125/d$
$L_{(25)} = 354.48$ mg/l
$L_{(20)} = 322.25$ mg/l
$Y_{5(20)} = 220.34$ mg/l

**<u>Determination of BOD from lab data</u>**

The following observations were made in the laboratory on 2% dilution of waste water :

D.O. of aerated dilution water - 7 mg/lt
D.O. of diluted sample after 5 days of incubation - 2 mg/lt
D.O. of original sample of waste water - 0.5 mg/lt.
Calculate the 5 day BOD of the sample and the ultimate first stage BOD, assuming The deoxygenation rate constant at 20°C as 0.1. The test was conduct at 20°C.

**Answer:**

**5 days 20°C BOD=($DO_b$ – $DO_i$) 100/% - ($DO_b$ – $DO_s$)**

$DO_b$ = Dissolved oxygen in blank solution = 7 mg/l

$DO_i$ = Dissolved oxygen in diluted sample after 5 days= 2 mg/l

$DO_s$ = Dissolved oxygen undiluted wastewater sample = 0.5 mg/l

5 days 20°C BOD= (7 – 2) 100/2 - (7 – 0.5)

**5 days 20°C BOD = 243.5 mg/l**

$Y_{5(20)} = L_{(20)} ( 1 – 10^{(– K(20) \times t5)} )$
$243.5 = L_{(20)} ( 1 – 10^{(– 0.1) \times 5)} )$

**Ultimate first stage BOD at 20$^0$C = 356.11 mg/l**

**Chemical oxygen demand (COD)**
It is the oxygen equivalent of organic matter. It is determined by measuring the dissolved oxygen used during the chemical oxidation of organic matter in 3 hours.
**Total organic carbon (TOC)-**This method measures the organic carbon existing in the wastewater by injecting a sample of the WW in special device in which the carbon is oxidized to carbon dioxide then carbon dioxide is measured and used to quantify the amount of organic matter in the WW. This method is only used for small concentration of organic matter.
**Theoretical oxygen (ThOD)-**If the chemical formula of the organic matter existing in the WW is known the ThOD may be computed as the amount of oxygen needed to oxidize the organic carbon to carbon dioxide and other end products.

**Difference between BOD and COD**
BOD (Biochemical oxygen demand) - The amount of oxygen required by micro-organisms to degrade the organic matter and can be calculated as BOD of diluted and undiluted samples. The BOD value depends on the dissolved organic matter in the waste water samples. More the organic matter, more the demand of oxygen by microbes to degrade it. Whereas as COD (Chemical Oxygen Demand) - In this process , Use of strong chemical agent (such as potassium dichromate) is done to degrade both the organic as well as inorganic matter present in the wastewater samples. Also, COD values are always higher than the BOD values, because COD includes both bio-degradable and non-biodegradable substances whereas BOD contains only bio-degradable.

**Inorganic Matter**-The following are the main inorganic materials of concern in wastewater treatment:

1. **Chlorides:-**
   - High concentrations indicate that the water body has been used for waste disposal.
   - It affects the biological process in high concentrations.

**2. Nitrogen:-**TKN = Total Kjeldahl nitrogen.
= Organic Nitrogen + ammonia Nitrogen (120 mg/l).

**3. Phosphorus:-**

- Municipal waste contains (4-15 mg/l).

**4. Sulfur:-**

* Sulfate exists in waste and necessary for synthesis of proteins.

$$\text{Organic matter} + SO_4^{-2} \rightarrow S^{-2} + H_2O + CO_2$$

$$S^{-2} + 2H^{+} \rightarrow H_2S$$

**5. Toxic inorganic Compounds:-**
Copper, lead, silver, chromium, arsenic, boron.

**6. Heavy metals:-**
Nickels, Mn, Lead, chromium, cadmium, zinc, copper, iron mercury.

**Gases:-**The following are the main gases of concern in wastewater treatment: $N_2$, $O_2$, $CO_2$, $H_2S$, $NH_3$, $CH_4$

**pH:-**The hydrogen-ion concentration is an important parameter in both natural waters and wastewaters. It is a very important factor in the biological and chemical wastewater treatment. Water and wastewater can be classified as neutral, alkaline or acidic according to the following ranges:

PH = 7 neutral.
PH > 7 Alkaline.
PH < 7 Acidic.

**Biological Characteristics:-**
The environmental engineer must have considerable knowledge of the biological of waste water because it is a very important characteristics factor in wastewater treatment. The Engineer should know:-

1. The principal groups of microorganisms found in wastewater.
2. The pathogenic organisms.
3. Indicator organisms (indicate the –presence of pathogens).
4. The methods used to amount the microorganisms.
5. The methods to evaluate the toxicity of treated wastewater .

**Main groups of Microorganisms:-**
The main microorganisms of concern in wastewater treatment are Bacteria, Fungi, Algae, Protozoa, Viruses, and pathogenic microorganisms groups.

**Bacteria:-**Types: Spheroid, rod curved rod, spiral, filamentous. Some important bacteria:-
Pseudomonas:-reduce NO3to N2, So it is very important in biological nitrate removal in treatment works.

Zoogloea:-helps through its slime production in the formation of flocs in the aeration tanks.
Sphaerotilus natuns: Causes sludge bulking in the aeration tanks.

Bdellovibrio: destroy pathogens in biological treatment.

Acinetobacter: Store large amounts of phosphate under aerobic conditions and release it under an –anaerobic condition so, they are useful in phosphate removal.

Nitrosomonas: transform NH4into NO2-
Nitrobacter: transform NO2-to NO3-
Coliform bacteria:-The most common type is E-Coli or Echerichia Coli, (indicator for the presence of pathogens). E-Coli is measured in (No/100mL)

**Fungi:**

•Important in decomposing organic matter to simple forms.

**Algae:**

• Cause eutrophication phenomena. (Negative effect) • Useful in oxidation ponds. (Positive effect) • Cause taste and problems when decayed. (Negative effect)

**Protozoa:**

•Feed on bacteria so they help in the purification of treated waste water. •Some of them are pathogenic.

**Viruses:**

Viruses are a major hazard to public health. Some viruses can live as long as 41 days in water and wastewater at 20$^{o}$ C. They cause lots of dangerous diseases.

**Pathogenic organisms:**

The main categories of pathogens are:- Bacteria, Viruses, protozoa, helminthes

Typical Wastewater Composition

| | Concentration | | | |
|---|---|---|---|---|
| Contaminants | Unit | Weak | Medium | Strong |
| Solids, total (TS) | mg/L | 350 | 720 | 1200 |
| Dissolved, total (TDS) | mg/L | 250 | 500 | 850 |
| Fixed | mg/L | 145 | 300 | 525 |
| Volatile | mg/L | 105 | 200 | 325 |
| Settle able solids (SS) | mg/L | 100 | 220 | 350 |
| Fixed | mg/L | 20 | 55 | 75 |
| Volatile | mg/L | 80 | 165 | 275 |
| Settle able Solids | mg/L | 5 | 10 | 20 |
| Biochemical oxygen demand, mg/l: | | | | |
| 5-day, 20 ˚C (BOD5,20 C) | mg/L | 110 | 220 | 400 |
| Total organic carbon (TOC) | | 80 | 160 | 290 |
| Chemical oxygen demand (COD) | mg/L | 250 | 500 | 1000 |
| Nitrogen (total as N) | mg/L | 20 | 40 | 85 |
| Organic | mg/L | 8 | 15 | 35 |
| Free ammonia | mg/L | 12 | 25 | 50 |
| Nitrites | mg/L | 0 | 0 | 0 |

| | | | | |
|---|---|---|---|---|
| Nitrites | mg/L | 0 | 0 | 0 |
| Phosphorus (total as P) | mg/L | 4 | 8 | 15 |
| Organic | mg/L | 1 | 3 | 5 |
| Inorganic | mg/L | 3 | 5 | 10 |
| Chloridesa | mg/L | 30 | 50 | 100 |
| Sulfatea | mg/L | 20 | 30 | 50 |
| Alkalinity (as CaCO3) | mg/L | 50 | 100 | 200 |
| Grease | mg/L | 50 | 100 | 150 |
| Total coliformb | no/100 ml | 106-107 | 107–108 | 107–109 |
| Volatile organic compounds (VOCs) | Mg/L | <100 | 100 -400 | > 400 |

**Population equivalent:**

**Population equivalent** or **unit per capita loading**, (**PE**), in waste-water treatment is the number expressing the ratio of the sum of the pollution load produced during 24 hours by industrial facilities and services to the individual pollution load in household sewage produced by one person in the same time.

For practical calculations, it is assumed that one unit equals to 54 grams of BOD per 24 hours.

$$PE = \frac{BOD\ load\ from\ industry\ \left[\frac{kg}{day}\right]}{0.054\ \left[\frac{kg}{inhab \cdot day}\right]}$$

## Anaerobic and aerobic activity

**Microorganisms** , like all living things, **require food for growth** . Biological sewage treatment consists of a step-by-step, continuous, sequenced attack on the organic compounds found in wastewater and upon which the microbes feed.

**Aerobic Decomposition**

A biological process, in which, organisms use available organic matter to support biological activity. The process uses organic matter, nutrients, and dissolved oxygen, and produces stable solids, carbon dioxide, and more organisms. The microorganisms which can only survive in aerobic conditions are known as aerobic organisms. In sewer lines the sewage becomes anoxic if left for a few hours and becomes anaerobic if left for more than 1 1/2 days. Anoxic organisms work well with aerobic and anaerobic organisms. Facultative and anoxic are basically the same concept.

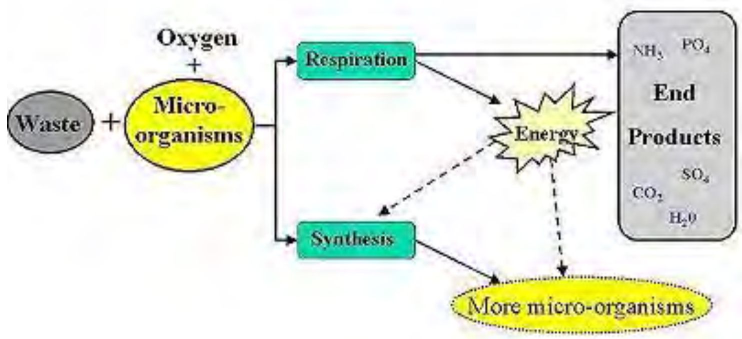
**Anaerobic Decomposition**

A biological process, in which, decomposition of organic matter occurs without oxygen. Two processes occur during anaerobic decomposition. First, facultative acid forming bacteria use organic matter as a food source and produce volatile (organic) acids, gases such as carbon dioxide and hydrogen sulfide, stable solids and more facultative organisms. Second, anaerobic methane formers use the volatile acids as a food source and produce methane gas, stable solids and more anaerobic methane formers. The methane gas produced by the process is usable as a fuel. The methane former works slower than the acid former, therefore the pH has to stay constant consistently, slightly basic, to optimize the creation of methane. You need to constantly feed it sodium bicarbonate to keep it basic.

**Aerobic Digestion**

Aerobic digestion of waste is the natural biological degradation and purification process in which bacteria that thrive in oxygen-rich environments break down and digest the waste.

During oxidation process, pollutants are broken down into carbon dioxide ($CO_2$ ), water ($H_2O$), nitrates, sulphates and biomass (microorganisms). By operating the oxygen supply with **aerators,** the process can be significantly accelerated. Of all the biological treatment methods, aerobic digestion is the most widespread process that is used throughout the world.




**Biological and chemical oxygen demand**

**Aerobic bacteria** demand oxygen to decompose dissolved pollutants. Large amounts of pollutants require large quantities of bacteria; therefore the demand for oxygen will be high.

**The Biological Oxygen Demand (BOD)** is a measure of the quantity of dissolved organic pollutants that can be removed in biological oxidation by the bacteria. It is expressed in **mg/l.**

**The Chemical Oxygen Demand (COD)** measures the quantity of dissolved organic pollutants than can be removed in chemical oxidation, by adding strong acids. It is expressed in **mg/l.**

The **BOD/COD** gives an indication of the fraction of pollutants in the wastewater that is biodegradable.

**Advantages of Aerobic Digestion**

**Aerobic bacteria** are very efficient in breaking down waste products. The result of this is; aerobic treatment usually yields better effluent quality that that obtained in anaerobic processes. The aerobic pathway also releases a substantial amount of energy. A portion is used by the microorganisms for synthesis and growth of new microorganisms.

**Anaerobic Digestion**

Anaerobic digestion is a complex biochemical reaction carried out in a number of steps by several types of microorganisms that require little or no oxygen to live. During this process, a gas that is mainly composed of methane and carbon dioxide, also referred to as biogas, is produced. The amount of gas produced varies with the amount of organic waste fed to the digester and temperature influences the rate of decomposition and gas production.

**Anaerobic digestion occurs in four steps:**

• **Hydrolysis** : Complex organic matter is decomposed into simple soluble organic molecules using water to split the chemical bonds between the substances.

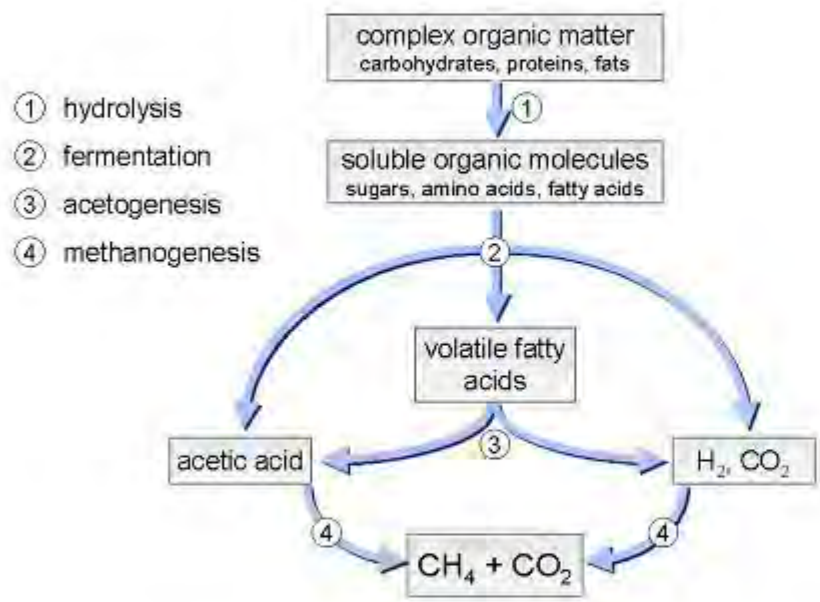
• **Fermentation or Acidogenesis:** The chemical decomposition of carbohydrates by enzymes, bacteria, yeasts, or molds in the absence of oxygen.

• **Acetogenesis:** The fermentation products are converted into acetate, hydrogen and carbon dioxide by what are known as acetogenic bacteria.

• **Methanogenesis:** Is formed from acetate and hydrogen/carbon dioxide by methanogenic bacteria.

The acetogenic bacteria grow in close association with the methanogenic bacteria during the fourth stage of the process. The reason for this is that the conversion of the fermentation products by the acetogens is thermodynamically only if the hydrogen concentration is kept sufficiently low. This requires a close relationship between both classes of bacteria.

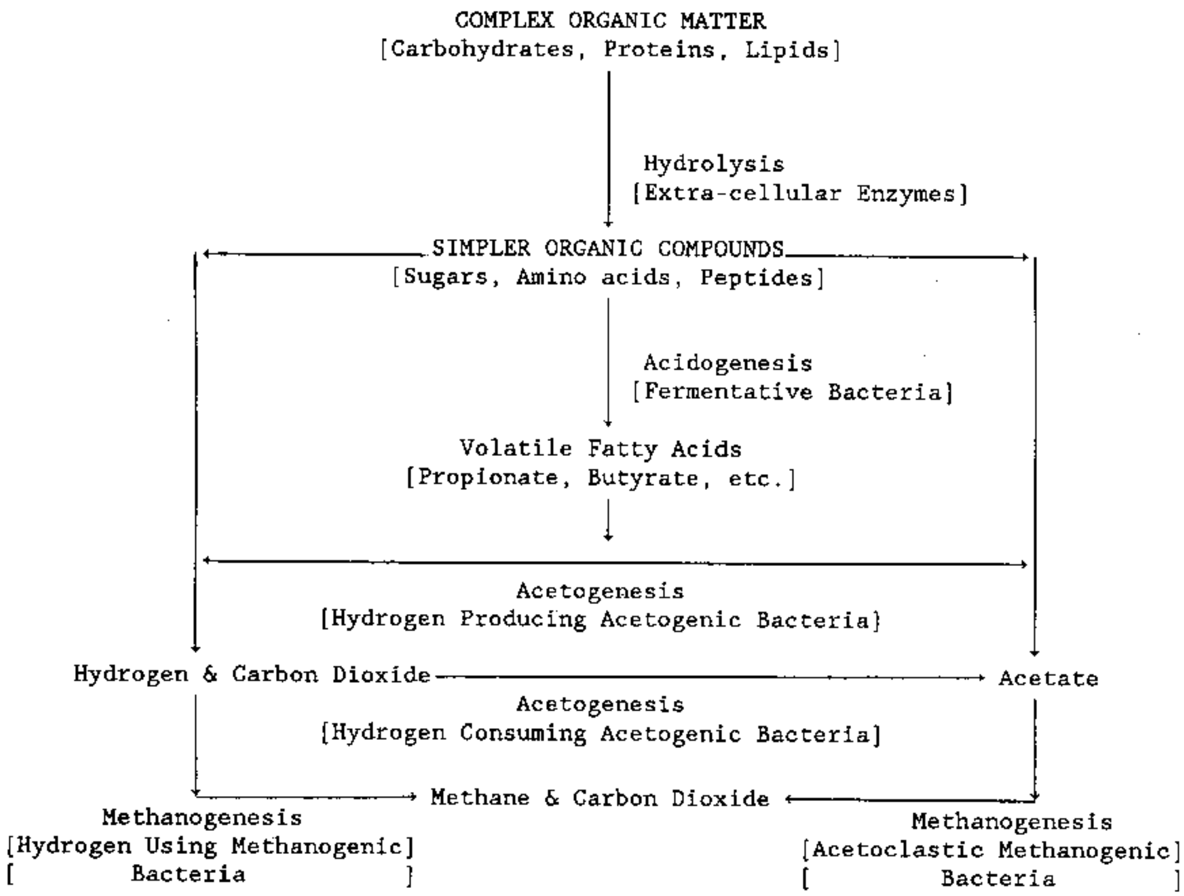
The anaerobic process only takes place under strict anaerobic conditions. It requires specific adapted bio-solids and particular process conditions, which are considerably different from those needed for aerobic treatment.



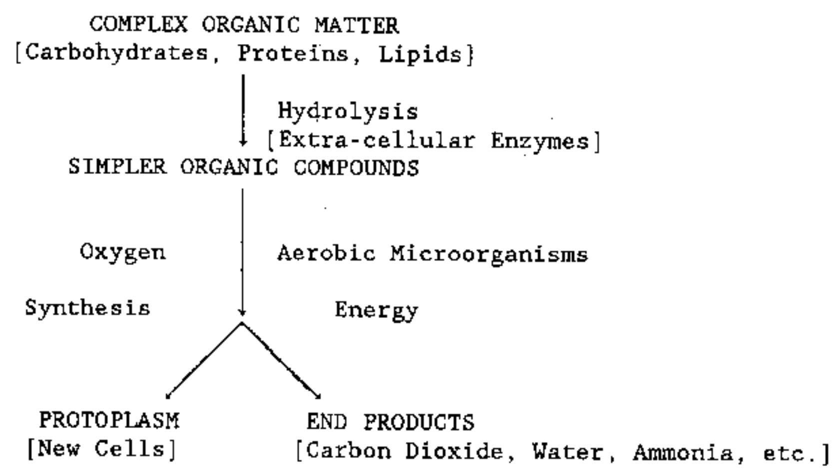

**Advantages of Anaerobic Digestion**

Wastewater pollutants are transformed into methane, carbon dioxide and smaller amount of bio-solids. The biomass growth is much lower compared to those in the aerobic processes. They are also much more compact than the aerobic bio-solids.

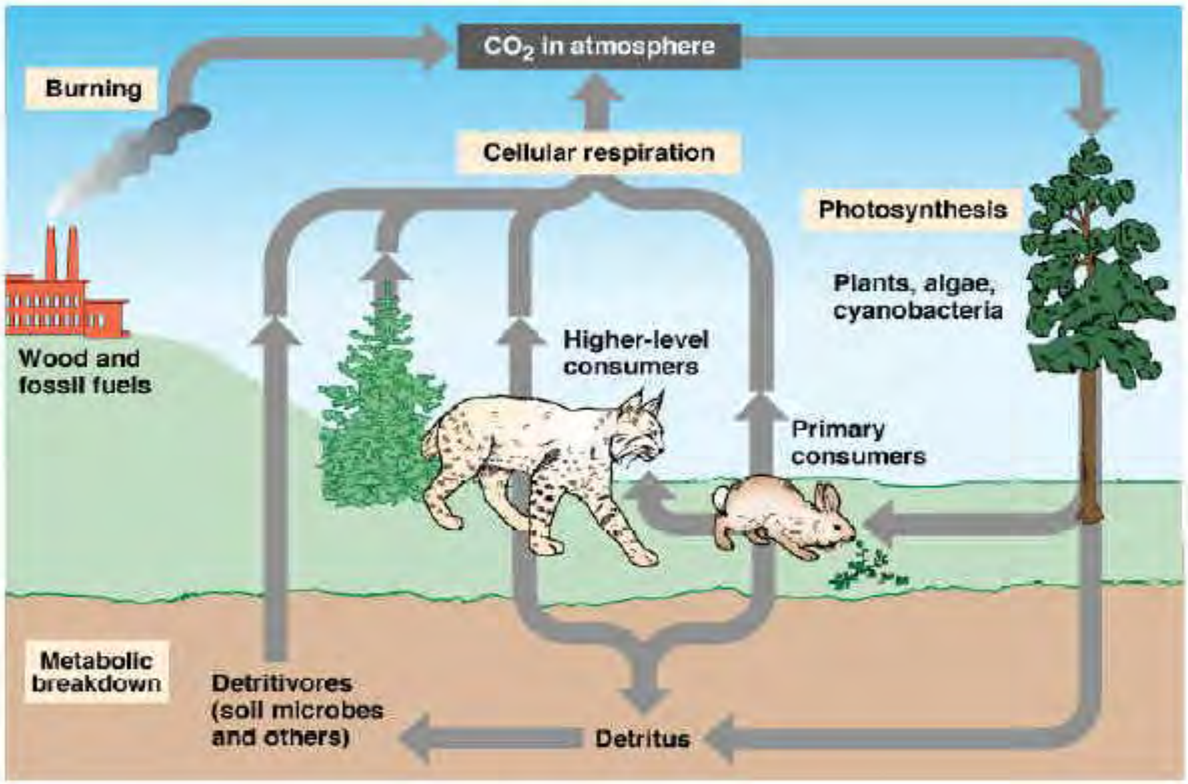
## ANAEROBIC TREATMENT CONVERSIONS




## AEROBIC TREATMENT CONVERSIONS



## CARBONCYCLE



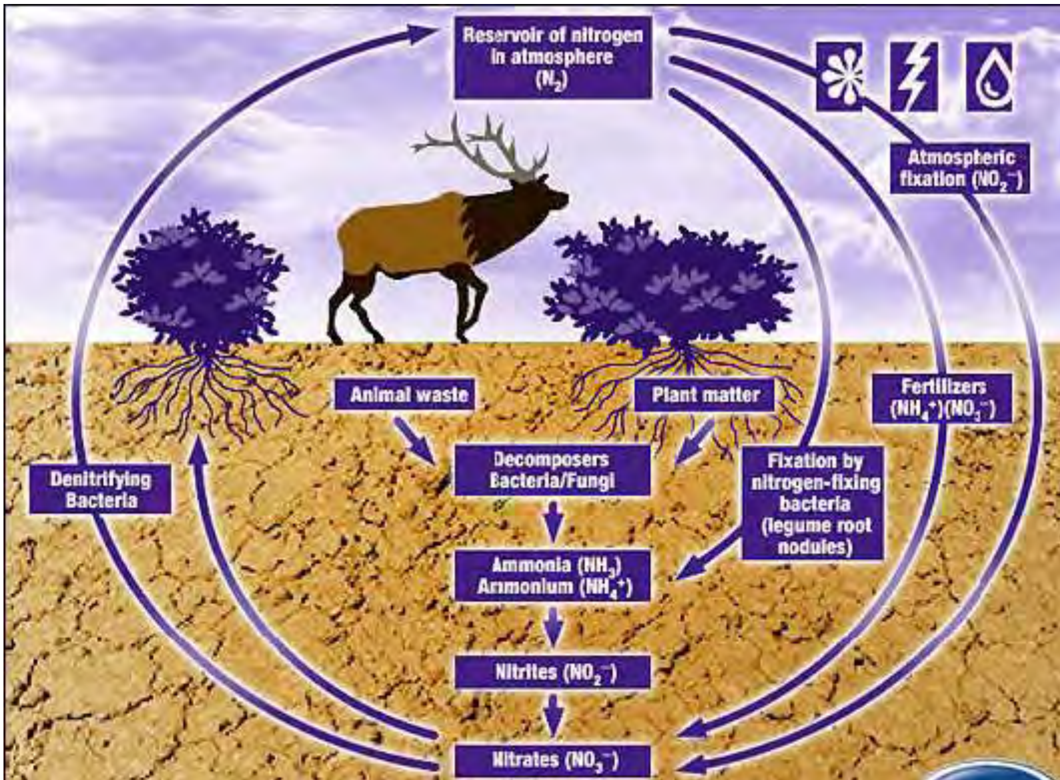

**I. Carbon EXISTS in abiotic environment as:**
1. Carbon dioxide [$CO_2$ (gas)] in the atmosphere dissolves in $H_2O$ to form $HCO_3$-
2. Carbonate rocks (limestone & coral = $CaCO_3$)
3. Deposits of coal, petroleum, and natural gas derived from once living things
4. Dead organic matter (humus in the soil)

**II. Carbon ENTERS biotic environment through:**
1. Photosynthesis: changes light energy to chemical energy

**III. Carbon RETURNS to atmosphere by:**
1. Respiration - $CO_2$
2. Decomposition / Decay
3. Burning

**IV. Carbon Cycle and Humans:**
1. Removal of photosynthesizing plants
2. Combustion of fossil fuels

## NITROGEN CYCLE




* ~79% of air is $N_2$ gas
* N is essential to plants and animals
* Plants and animals can't use $N_2$ gas
* Usable N: ammonia ($NH_3$) or nitrate ($NO_3^-$)

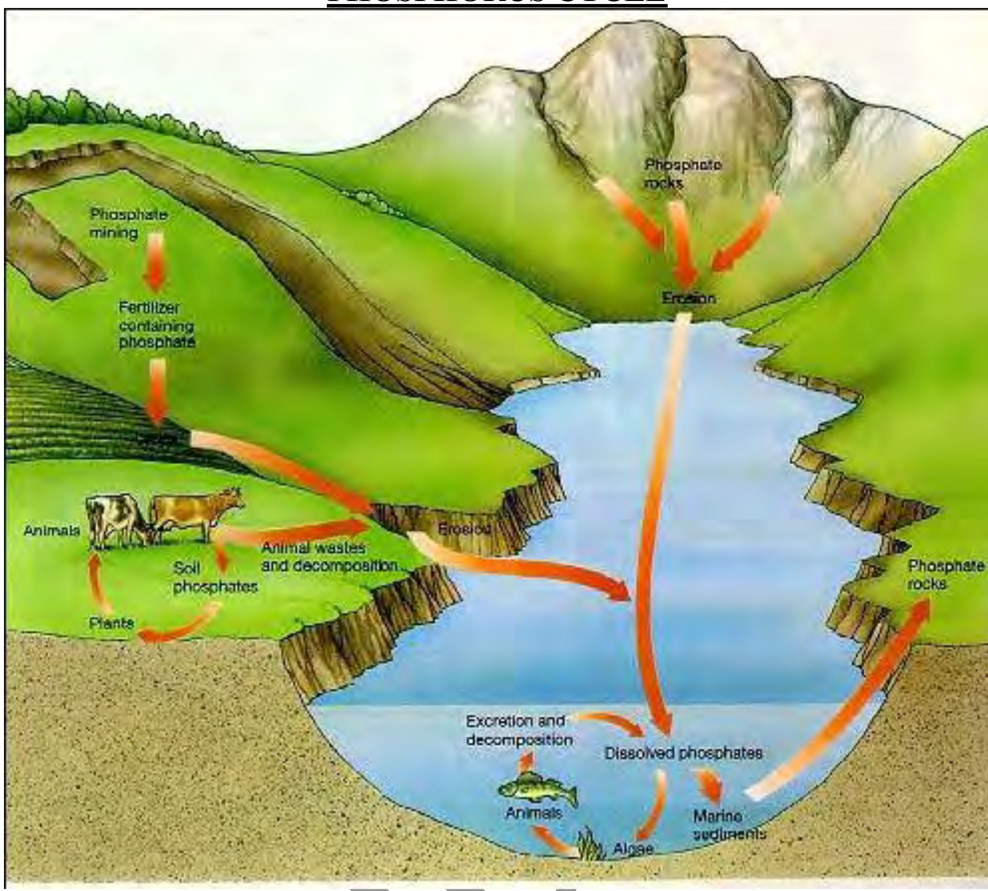
**I. Conversion of atmospheric $N_2$ to $NH_3$ and $NO_3^-$:**
*Nitrogen fixation*
1. Aquatic ecosystems: blue-green algae
2. Terrestrial ecosystems: bacteria on root nodules of legumes (peas, beans, alfalfa, clover)
3. Lightening

**II. Nitrogen RETURNS to soil by:**
1. Decomposition of once living things
   ammonifying bacteria + fungi
2. Exists in soil as nitrate ($NO_3^-$), nitrite ($NO_2^-$), and ammonia ($NH_3$)

**III. Nitrogen returns to atmosphere by:**
1. Denitrifying bacteria

**IV. Nitrogen Cycle and Humans:**
1. Nitrogen required for genetic materials (DNA, RNA, amino acids)

## PHOSPHORUS CYCLE




Major environmental reservoir: rocks
1. Leaching: water dissolves phosphates in rocks and carries to lake, stream, etc.
2. Dissolved phosphate: used by plants and passed through food chain
3. Animals return phosphorus to environment by:
i. excretion
ii. death and decay

**Phosphorus Cycle and Humans:**
1. Phosphates mined for fertilizers - returns P to soil
2. Erosion: P in soil and rocks washed away into water systems

# PART - B

## Unit-V

### DISPOSAL OF EFFLUENTS

#### OBJECTS OF SEWAGE DISPOSAL

- To eliminate or reduce danger to the public health by possible contamination of water supplies.
- To render the sewage inoffensive without causing odour or nuisance.
- To prevent the life of fish or other aquatic life by allowing raw sewage into bodies of water as such.
- The destruction of fish & other Aquatic life can be prevented by the swage disposal methods.
- With proper sewage disposal the environment or the areas does not become polluted.
- Sanitary conditions are maintained in the area.

There are two principal methods of sewage disposal by utilizing natural agencies i.e.,

1. Dilution i.e., disposal of sewage of water.
2. Land disposal or irrigation.

Both methods are very simple. But these may be regulated carefully so that the quantity of sewage put in water or applied to land is such that they are capable of receiving the organic load present in the effluent.

#### Dilution

Dilution is the disposal of sewage by discharging it into large bodies of water like sea, streams, rivers etc. This method is possible only when the natural water is available in large quantity near the town. Proper care should be taken while discharging sewage in water so that sewage may not pollute natural water and make it unfit for any other purposes like bathing, drinking, irrigation etc.

#### CONDITIONS FAVOURABLE FOR DILUTION

1. Where sewage is fresh.
2. Where favorable currents exits in a stream.
3. Where sewage is almost free from floating/ settleable solids.
4. Where thorough mixing is possible.
5. Where diluting water has high quantities of dissolved oxygen.
6. When the city is situated near river or sea.

**Self purification phenomenon:**

**SELF PURIFICATION OF NATURAL STREAMS**

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. The amount of dissolved Oxygen (DO) in water is one of the most commonly used indicators of a river health. As DO drops below 4 or 5 mg/L the forms of life that can survive begin to be reduced. A minimum of about 2.0 mg/L of dissolved oxygen is required to maintain higher life forms. A number of factors affect the amount of DO available in a river. Oxygen demanding wastes remove DO; plants add DO during day but remove it at night; respiration of organisms removes oxygen. In summer, rising temperature reduces solubility of oxygen, while lower flows reduce the rate at which oxygen enters the water from atmosphere.

**Factors Affecting Self Purification**

**1. Dilution**: When sufficient dilution water is available in the receiving water body, where the waste water is discharged, the DO level in the receiving stream may not reach to zero or critical DO due to availability of sufficient DO initially in the river water before receiving discharge of wastewater.
**2. Current**: When strong water current is available, the discharged wastewater will be thoroughly mixed with stream water preventing deposition of solids. In small current, the solid matter from the wastewater will get deposited at the bed following decomposition and reduction in DO.
**3. Temperature**: The quantity of DO available in stream water is more in cold temperature than in hot temperature. Also, as the activity of microorganisms is more at the higher temperature, hence, the self-purification will take less time at hot temperature than in winter.
**4. Sunlight:** Algae produces oxygen in presence of sunlight due to photosynthesis. Therefore, sunlight helps in purification of stream by adding oxygen through photosynthesis.
**5. Rate of Oxidation:** Due to oxidation of organic matter discharged in the river DO depletion occurs. This rate is faster at higher temperature and low at lower temperature. The rate of oxidation of organic matter depends on the chemical composition of organic matter.

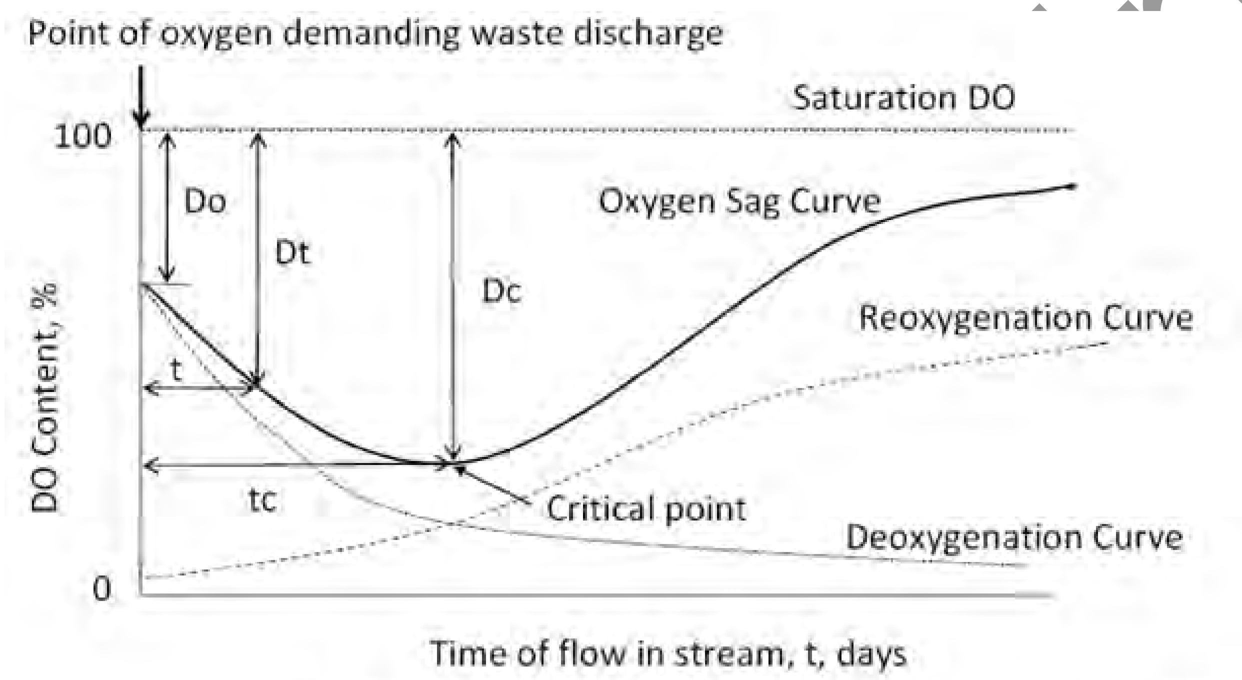
**Oxygen Sag Curve**

The oxygen sag or oxygen 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. The amount of resultant oxygen deflect can be obtained by algebraically adding the de - oxygenation and re -oxygenation curves. The resultant curve so obtained is called oxygen sag curve

Oxygen deficit, D = Saturation DO – Actual DO

The saturation DO value for fresh water depends upon the temperature and total dissolved salts present in it; and its value varies from 14.62 mg/L at 0oC to 7.63 mg/L at 30oC, and lower DO at higher temperatures.

The DO in the stream may not be at saturation level and there may be initial oxygen deficit ‗Do'. At this stage, when the effluent with initial BOD load Lo, is discharged in to stream, the DO content of the stream starts depleting and the oxygen deficit (D) increases. The variation of oxygen deficit (D) with the distance along the stream, and hence with the time of flow from the point of pollution is depicted by the ‗Oxygen Sag Curve' . The major point in sag analysis is point of minimum DO, i.e., maximum deficit. The maximum or critical deficit (Dc) occurs at the inflexion points (as shown in fig) of the oxygen sag curve.




Deoxygenation. reoxygenation and oxygen sag curve

**Deoxygenation and Reoxygenation Curves**

**De-oxygenation curve:** The curve which represents (or) showing the depletion of D.O with time at the given temperature.
**Re-oxygenation Curve**: In order to counter balance the consumption of D.O due to the de – oxygenation, atmosphere supplies oxygen to the water and the process is called the re - oxygenation
When wastewater is discharged in to the stream, the DO level in the stream goes on depleting. This depletion of DO content is known as deoxygenation. The rate of deoxygenation depends upon the amount of organic matter remaining (Lt), to be oxidized at any time t, as well as temperature (T) at which reaction occurs. The variation of depletion of DO content of the stream with time is depicted by the deoxygenation curve in the absence of aeration. The ordinates below the deoxygenation curve (Figure 12.1) indicate the oxygen remaining in the natural stream after satisfying the bio-chemical demand of oxygen. When the DO content of the stream is gradually consumed due to BOD load, atmosphere supplies oxygen continuously to the water, through the process of re-aeration or reoxygenation, i.e., along with deoxygenation, re-aeration is continuous process.

The rate of reoxygenation depends upon:
i) Depth of water in the stream: more for shallow depth.
ii) Velocity of flow in the stream: less for stagnant water.
iii) Oxygen deficit below saturation DO: since solubility rate depends on difference between saturation concentration and existing concentration of DO.
iv) Temperature of water: solubility is lower at higher temperature and also saturation concentration is less at higher temperature.

## Zones of Purification

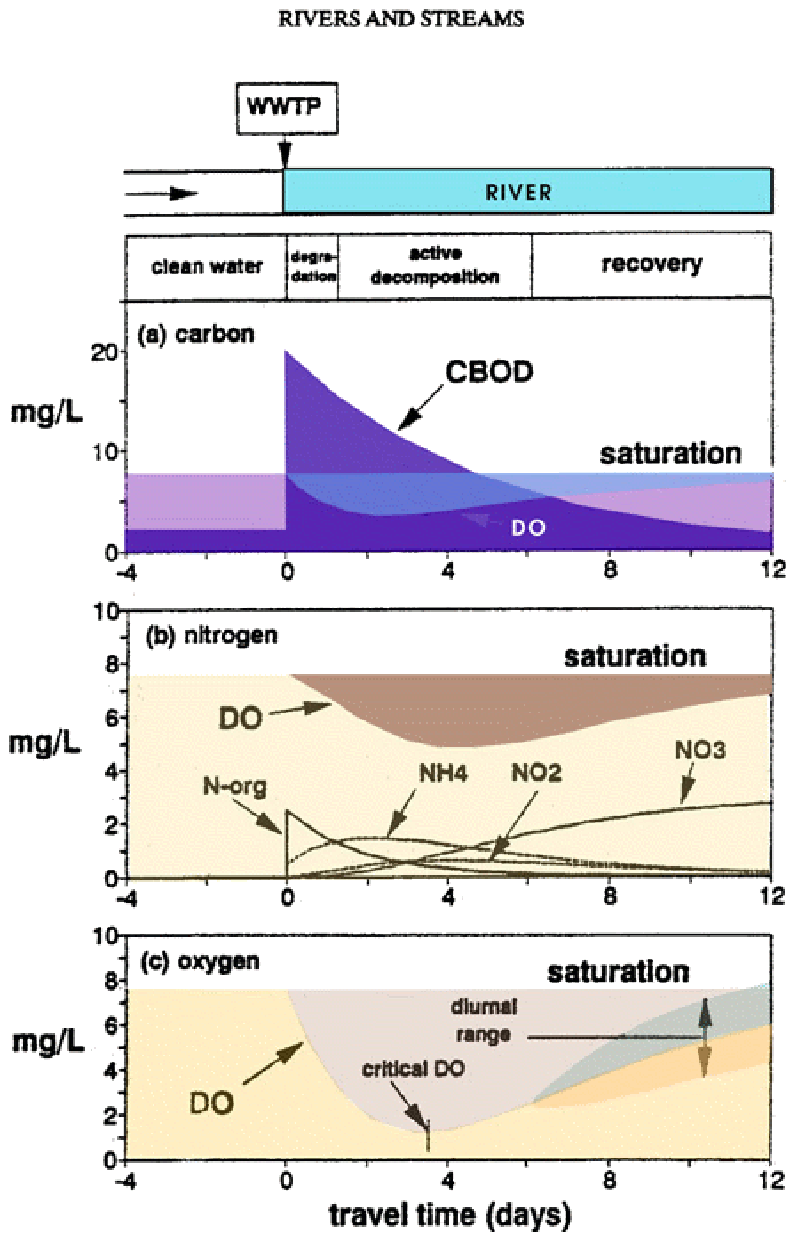
When sewage is discharged into water, a succession of changes in water quality takes place. If the sewage is emptied into a lake in which currents about the outfall are sluggish and shift their direction with the wind, the changes occur in close proximity to each other and, as a result, the pattern of changes is not crisply distinguished. If, on the other hand, the water moves steadily away from the outfall, as in a stream, the successive changes occur in different river reaches and establish a profile of pollution which is well defined. However, in most streams, this pattern is by no means static. It shifts longitudinally along the stream and is modified in intensity with changes in season and hydrography.

When a single large charge of sewage is poured into a clean stream, the water becomes turbid, sunlight is shut out of the depths, and green plants, which by photosynthesis remove carbon dioxide from the water and release oxygen to it, die off. Depending on the stream velocity, the water soon turns nearly black. Odorous sulfur compounds are formed and solids settle to the bottom, forming a sludge. The settled solids soon decompose, forming gases such as ammonia, carbon dioxide, and methane or marsh gas. Scavenging organisms increase in number until they match the food supply. The oxygen resources are drawn upon heavily and, when overloaded, become exhausted. Life in such waters is confined to anaerobic bacteria (which exist when no oxygen is available), larvae of certain insects such as mosquitoes, and a few worms. There are no fish; turtles are generally the only forms of higher life present. This condition is known as the **zone of degradation**.

In a second zone, or **zone of decomposition**, more solids settle out, the water becomes somewhat clearer, and sunlight penetrates the surface. Oxygen is absorbed from the atmosphere at the air-water interface permitting the establishment of aerobic (oxygen available) conditions. The aerobic bacteria continue the conversion of organic matter into nitrates, sulfates. and carbonates. These, together with the carbon dioxide produced by decomposition as well as by bacteria and plant life, are food sources. With sunlight now penetrating the water, and with abundant food, algae begin to flourish and form a green scum over the surface.

In the third zone, or **zone of recovery**, algae become more numerous and self-purification proceeds more rapidly. Green plants utilizing carbon dioxide and oxygen will liberate in the say time more oxygen than is consumed, thus hastening the recovery of the stream. Simultaneously, the fish that require little oxygen such as catfish and carp, are also found. As the dissolved oxygen increases, more types of fish appear. After recovery, in the **zone of cleaner water**, fish find the stream highly favorable, as the algae support various aquatic insects and other organisms on which fish feed. The water is clear or turbid according to concentration of algae, and may have odor for the same reason.

Throughout the stages of recovery of self-purification, disease organisms are greatly reduced in number because they lack proper food, and experience unfavorable temperatures and pH values of water. However, the water is still dangerous since all disease organisms have not perished,



**BOD of the resulting mixture:**

At the outfall, BOD of the river/wastewater mixture ($L_0$) is given by:

$$L_0 = \frac{Q_r L_r + Q_w L_w}{Q_r + Q_w}$$

Where :

$L_0$ = Ultimate BOD at the point of waste discharge

$Q_r$ = Flow in the river upstream of the discharge

$L_r$ = Ultimate BOD of the river water

$Q_w$ = Flow of wastewater from the discharge

$L_w$ = Ultimate BOD in the discharged wastewater

**Problem:**

*A wastewater effluent of 600 l/s with a BOD = 60 mg/l, DO = 2.5 mg/l and temperature of 25°C enters a river where the flow is 30 m³/sec and BOD = 3 mg/l, DO = 8.5 mg/l and temperature of 16°C. Deoxygenation constant for the waste is 0.10 per day at 20°C. The velocity of water in the river downstream is 0.15 m/s and depth of flow is 1.5 m. Determine the following after mixing of wastewater with the river water : (i) combined discharge. (ii) BOD (iii) DO, and (iv) Temperature*

**Answer:**

Wastewater discharge = 600 l/s = 0.6 m³/s.

(*i*) Combined discharge $= Q_R + Q_E = 30 + 0.6 = 30.6\ \text{m}^3/\text{s}$

(*ii*) $(\text{BOD})_{mix} = \dfrac{(30 \times 3) + (0.6 \times 60)}{30 + 0.6} = 4.118\ \text{mg/l}$

(*iii*) $(\text{DO})_{mix} = \dfrac{(30 \times 8.5) + (0.6 \times 2.5)}{30 + 0.6} = 8.382\ \text{mg/l}$

(*iv*) $(\text{Temp.})_{mix} = \dfrac{(30 \times 16) + (0.6 \times 2.5)}{30 + 0.6} = 16.18°\text{C}$

**Problem:**
Sewage from a town is discharged into a river having a discharge of 250 lit/sec. If the quantity of sewage is 9 MLD and the BOD of sewage and river are 260 mg/l and 6 mg/l respectively, determine the BOD of the diluted water. If it is required to reduce the BOD of the diluted water to 20 mg/l, what should be the discharge in the river?

**Answer:**

I case
BOD of diluted water, $C_{BOD}$ = 80.705 mg/l
II case
Discharge in the river, $Q_R$ = 154.28 MLD

**Problem**

Sewage from a town is discharged into a river having a discharge of 200 l/sec. If the quantity of sewage is 9 MLD and the BOD of sewage and river are 260 mg/l and 6 mg/l respectively, determine the BOD of the diluted water. If it is required to reduce the BOD of the diluted water to 20 mg/l, what should be the discharge in the river?

**Answer:**

River discharge, $Q_R$ = 17.28 MLD

I. Concentration of BOD of the diluted water, $C_{BOD}$ = 92.98mg/l

II. Discharge in the river, $Q_R$ = 154.28 MLD

**Problem:**

Sewage from a town is discharged into a river having a discharge of 250 l/sec. If the quantity of sewage is 10 MLD and the BOD of sewage and river are 300 mg/l and 10 mg/l respectively, determine the BOD of the diluted water. If it is required to reduce the BOD of the diluted water to 20 mg/l, what should be the discharge in the river?

**Answer:**

I case
River discharge, $Q_R$ = 21.6 MLD
BOD of diluted water, $C_{BOD}$ = 101.77 mg/l
II case
Discharge in the river, $Q_R$ = 280 MLD

**Problem**

A city discharges 100 cumecs of sewage into 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/s. The 5 day BOD of sewage at 20°C is 280 mg/L. Find when and where the critical DO deficit will occur in the downstream portion of river. Also find the value of critical DO deficit Assume self-purification constant of river as 4.0, coefficient of de-oxygenation as 0.1 per day at 20°C and Saturation DO = 9.2 mg/L.

**Answer:**

i) DO mix = 8.625 mg/l
ii) Initial DO deficit= 9.2 – 8.625 = 0.58 mg/l
iii) 5 day BOD of the mix = 17.5 mg/l
iv) Ultimate BOD of the mix = 25.59 mg/l
v) Critical DO deficit = 4.12 mg/l
vi) Time at which critical DO deficit occurs = 1.905 days
vii) Distance downstream at which critical DO deficit occurs= 16.46 KM

**Sewage farming**

**ADVANTAGES AND DISADVANTAGES OF LAND DISPOSAL**

Advantages:-

1. Adds manure to land
2. Pollution of natural water courses is minimized.
3. Increase fertility of land.
4. Gives high calorific value to crops grown in sewage farms.
5. Does not require any installation of equipment involving high initial cost.
6. Crops could be grown and hence a return value is always possible to obtain.
7. Method specially suitable where large quantity of river water is not available at all times of the year.

DISADVANTAGES:-

1. Difficult to get land during rainy and harvest seasons.
2. Additional land is required for reserve.
3. Sanitary reasons may not permit growing of crops on sewage farms.
4. More land area is required is sewage volume is greater since land capacity is limited.
5. If all precautions are not taken, sewage farming results in sewage sickness to land and health to life.

## METHOD OF LAND TREATMENT

Sewage mainly contains water which can be used for irrigation purposes . The fertilising value of sewage is more because it contains n, potash & phosphate .The sewage can be applied in the following forms

- BROAD IRRIGATION
- SEWAGE FARMING.

**BROAD IRRIGATION**

In this method, sewage is allowed to flow over cultivated lands, from which a part of the sewage evaporates, some percolates and the rest escape into surface drainage channels. Sewage waters the land and adds to its fertilizing value, due to the presence of nitrogen, phosphorus, potash etc. These fertilizing elements of sewage are consumed by the roofs of crops. Crops like cotton, potatoes, sugarcane, grass etc, can be profitably grown. This is also called sewage farming.

**SEWAGE FARMING**

The process in which sewage is used for growing crops is known as sewage farming .The fertilising elements of sewage i. e nitrates, sulphates, & phosphates are used by the roots of crops. The nutrients of sewage make the fields fertile .It is a profitable business & a good income can be generated by sewage farming.

## 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**:- The area to be irrigated is divided into various parts surrounded by dykes. The sewage is filled like small ponds in between the dykes. The depth of dose varies from 3.0 cms. To 5.0 cms. Depending on the irrigation requirements.

**SURFACE IRRIGATION**:- This method is most suited in sloping area. Here, parallel drains are constructed in the fields. All these drains are connected to a distributaries drain with the help of regulating device so that sewage may flow in the require drain. Here when sewage flows over the fields, its large quantity is absorbed by the field and only excess quantity reaches another drain.

**ZIG ZAG METHOD**: In this method the ridges are arranged in a zag-zag method with corresponding furrows by their side

**LAGOONING**: These are used cheaply for sewage disposal. In this method the sewage is allowed to in a natural depression available or artificial constructed tanks. Detention period is about a month. During this period the sludge is stabilized and dried. The purified effluent passes way from an outlet placed at the other end. Lagoons should be shallow and must be constructed away from the town.

**SUB SURFACE IRRIGATION**: Here sewage is applied at the roofs of plants, through the open jointed agricultural drain-pipes. These pipes are laid about 1.0 m below the ground level. The sewage rises up due to capillary action. Here soil takes fewer loads but this is an economical method.

**BASIN METHOD**: In this method big trees are planted in an isolated manner, basins are formed around each tree. These basins are filled with sewage. This method is suitable for fruit gardens.

**RIDGE AND FERROW IRRIGATION:** In this method , sewage is supplied in furrows between crop rows. Sewage spreads laterally irrigating the area between two furrows. The width of furrow varies from 120-150 cm and the depth from 25-50 cm. The width of the ridge varies from 125-250 cm and length from 10-30 m. The percolated effluent is collected in underground drains flows towards natural drainage for disposal.

**Comparison of dilution & land treatment method of disposal of sewage**

| S.No. | Disposal by Dilution | Disposal by Land Treatment |
|---|---|---|
| 1. | Large volumes of water are required. | Large area of pervious soils are required. |
| 2. | Income can't be generated. | Income can be generated by adopting sewage farming. |
| 3. | It requires full or partial treatment. | It requires preliminary or primary treatment. |
| 4. | It causes stream pollution and an unavoidable health hazard. | It saves the inland streams from getting polluted by sewage. |
| 5. | The pollution travels down stream and thus the stream may lose its utility for various purposes such as bathing, recreation etc. | It returns to the land the fertilising element. |
| 6. | It requires no or low head pumping as the stream passes through lowest contours and valleys. | It requires high head pumping therefore the cost of pumping is high. |

**FAVOURABLE CONDITIONS FOR SUB-SURFACE IRRIGATION**

1. Subsoil water level is low.
2. Land is cheap.
3. Rainfall is less and irrigation water demand is heavy.
4. Large areas are available.
5. Where dilution water is not easily available.
6. Sub-surface strata is porous, favoring infiltration.
7. Climate is dry favoring drying up conditions.

**SEWAGE SICKNESS**

The phenomena of 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.

**Problem:**

. *A town, having a population of 50000 and the rate of water supply as 160 l/day, disposes off its sewage successfully by land treatment. The area of land available is 180 hectares. If 80% of the water supplied is converted into sewage, find out the consuming capacity of soil.*

**Solution.** Quantity of sewage $= 0.8 \times 50000 \times 160$

$= 6400 \times 10^3$ litres $= 6400\ m^3/day$

Total area of land $= 180$ hectares

Providing 50% exta area as reserve for rest and rotation,

Actual available area for land application $= \frac{180}{1.5} = 120$ hectares.

$\therefore$ Consuming capacity of soil $= \frac{6400}{120}$

$= 53.33$ cubic metres/hectare/day.

**Problem:**

A town with a population of 50,000 is provided with 180 lpcd of water. If no ground water is likely to enter sewers and 80% of water supplied goes as sewage, how much land including 50% reserve will be required for continuous broad irrigation, if the permitted loading is 1,34,000 l/d hec.

**Answer:**

Sewage flow, $Q_s = 7.2$ MLD
Land area required = 53.73 hectares
50% reserve area = 26.86 hectares
Total land area required = 80.59 hectares

**<u>Effluent disposal standards for land, surface and ocean</u>**

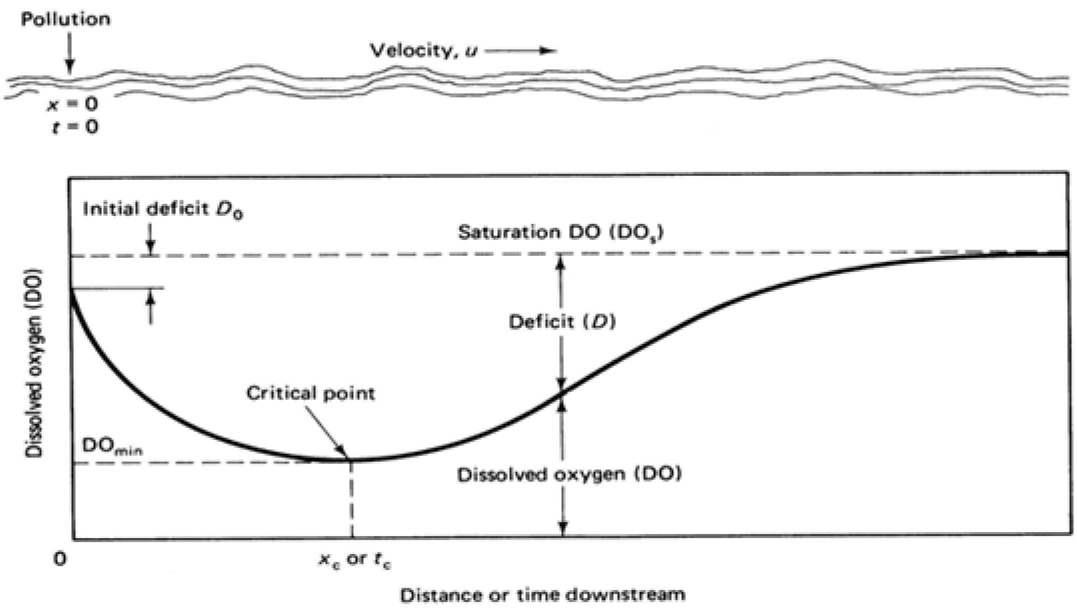
## [SCHEDULE – VI]
## GENERAL STANDARDS FOR DISCHARGE OF ENVIRONMENTAL POLLUTANTS PART-A : EFFLUENTS

| S. No. | Parameter | Standards | | | |
|---|---|---|---|---|---|
| | | Inland surface water | Public Sewers | Land for irrigation | Marine coastal areas |
| 1 | 2 | 3 | | | |
| | | (a) | (b) | (c) | (d) |
| 1. | Colour and odour | See 6 of Annexure-I | -- | See 6 of Annexure-I | See 6 of Annexure-I |
| 2. | Suspended solids mg/l, Max. | 100 | 600 | 200 | (a) For process waste water- 100<br>(b) For cooling water effluent 10 percent above total suspended matter of influent. |
| 3. | Particulate size of suspended solids | Shall pass 850 micron IS Sieve | -- | -- | (a) Floatable solids, max. 3 mm.<br>(b) Settleable solids, max. 850 microns. |
| 24. | *** | * | -- | *** | -- |
| 5. | pH Value | 5.5 to 9.0 | 5.5 to 9.0 | 5.5 to 9.0 | 5.5 to 9.0 |
| 6. | Temperature | shall not exceed 5°C above the receiving water temperature | -- | -- | shall not exceed 5°C above the receiving water temperature |
| 7. | Oil and grease mg/l Max. | 10 | 20 | 10 | 20 |
| 8. | Total residual chlorin mg/l Max. | 1.0 | -- | -- | 1.0 |
| 9. | Ammonical nitrogen (as N), mg/l Max. | 50 | 50 | -- | 50 |
| 10. | Total Kjeldahl Nitrogen (as $NH_3$) mg/l, Max. | 100 | -- | -- | 100 |
| 11. | Free ammonia (as $NH_3$) mg/l, Max. | 5.0 | -- | -- | 5.0 |
| 12. | Biochemical Oxygen demand [1][3 days at 27°C] mg/l max. | 30 | 350 | 100 | 100 |

| | | | | | |
|---|---|---|---|---|---|
| 13. | Chemical Oxygen Demand, mg/l, max. | 250 | -- | -- | 250 |
| 14. | Arsenic (as As), mg/l, max. | 0.2 | 0.2 | 0.2 | 0.2 |
| 15. | Mercury (as Hg), mg/l, Max. | 0.01 | 0.01 | -- | 0.01 |
| 16. | Lead (as Pb) mg/l, Max. | 0.1 | 1.0 | -- | 2.0 |
| 17. | Cadmium (as Cd) mg/l, Max. | 2.0 | 1.0 | -- | 2.0 |
| 18. | Hexavalent Chromium (as Cr+6), mg/l max. | 0.1 | 2.0 | -- | 1.0 |
| 19. | Total chromium (as Cr.) mg/l, Max. | 2.0 | 2.0 | -- | 2.0 |
| 20. | Copper (as Cu) mg/l, Max. | 3.0 | 3.0 | -- | 3.0 |
| 21. | Zinc (As Zn.) mg/l, Max. | 5.0 | 15 | -- | 15 |
| 22. | Selenium (as Se.) mg/l, Max. | 0.05 | 0.05 | -- | 0.05 |
| 23. | Nickel (as Ni) mg/l, Max. | 3.0 | 3.0 | -- | 5.0 |
| 1124. | * * * | * | * | * | * |
| 125. | * * * | * | * | * | * |
| 126. | * * * | * | * | * | * |
| 27. | Cyanide (as CN) mg/l Max. | 0.2 | 2.0 | 0.2 | 0.2 |
| 128. | * * * | * | * | * | * |
| 29. | Fluoride (as F) mg/l Max. | 2.0 | 15 | -- | 15 |
| 30. | Dissolved Phosphates (as P), mg/l Max. | 5.0 | -- | -- | -- |
| 231. | * * * | * | * | * | * |
| 32. | Sulphide (as S) mg/l Max. | 2.0 | -- | -- | 5.0 |
| 33. | | Phenoile compounds (as $C_6H_5OH$) mg/l, Max. | | | |
| 34. | | Radioactive materials : | | | |
| (a) Alpha emitter micro curie/ml. | 10-7 | 10-7 | | 10-8 | 10-7 |
| (b) Beta emitter micro curie/ml. | 10-6 | 10-6 | | 10-7 | 10-6 |
| 35. | Bio-assay test | 90% survival of fish after 96 hours in 100% effluent | 90% survival of fish after 96 hours in 100% effluent | 90% survival of fish after 96 hours in 100% effluent | 90% survival of fish after 96 hours in 100% effluent |

| | | | | | |
|---|---|---|---|---|---|
| 36. | Manganese (as Mn) | 2 mg/l | 2 mg/l | -- | 2 mg/l |
| 37. | Iron (as Fe) | 3 mg/l | 3 mg/l | -- | 3 mg/l |
| 38. | Vanadium (as V) | 0.2 mg/l | 0.2 mg/l | -- | 0.2 mg/l |
| 39. | Nitrate Nitrogen | 10 mg/l | -- | -- | 20 mg/l |
| 140. | * * * | * | * | * | * |

**Streeter Phelps Equation**




Streeter–Phelps oxygen-sag curve.

1. DO content is one of the most widely used indicators of overall ecological health of a body of water
   - fish need 4 to 5 mg/L to survive
   - under anaerobic conditions, undesirable (smelly) microbes can take over
   - many factors affect the DO level
2. If a river was healthy before we began discharging wastewater, a significant factor in its continued health or illness is the BOD added to it by wastewater.
   a. At the outfall, BOD of the river/wastewater mixture ($L_0$) is given by:

$$L_0 = \frac{Q_r L_r + Q_w L_w}{Q_r + Q_w}$$

Where :

$L_0$ = Ultimate BOD at the point of waste discharge

$Q_r$ = Flow in the river upstream of the discharge

$L_r$ = Ultimate BOD of the river water

$Q_w$ = Flow of wastewater from the discharge

$L_w$ = Ultimate BOD in the discharged wastewater

b. BOD is comparable to what we have in our stoppered bottle at the beginning of our BOD test

- As time passes (ie, the water moves downstream) the oxygen content of the river water is consumed in just the same way oxygen is consumed in the test
- BOD ($L_t$) in a test bottle at time t is given by:

$$L_t = L_0 e^{-k_D t}$$

- This formula holds in the river too ($k_D$ is the deoxygenation constant; it can be adjusted for temperature using $k_T = k_{20\theta}{}^{T-20}$)
- Knowing an average velocity of flow, we can calculate the BOD for a given distance downstream

DO remaining depends both on the rate of deoxygenation and on the rate of reoxygenation or reaeration

- The rate of reaeration, $r_R$, is given by:

  $r_R = -k_R * D$ with

  $k_R$ = reaeration time constant

  $D$ = DO deficit = $DO_s$-DO

- The reaeration time constant can be estimated from Table , or calculated by:

  $k_{R,20°C} = 3.9u^{1/2}/H^{3/2}$

  u = average stream velocity

  H = average stream depth

**Table - Reaeration constants**

| Water body | Ranges of $k_R$ at 20°C, base e |
|---|---|
| Small ponds and backwaters | 0.1-0:23 |
| Sluggish streams and large lakes | 0.23-0.35 |
| Large streams of low velocity | 0.35-0.46 |
| Large streams of normal velocity | 0.46-0.69 |
| Swift streams | 0.69-1.15 |
| Rapids and waterfalls | Greater than 1.15 |

Source: Peavy, Rowe and Tchobanoglous, 1985

- To start with, the waste has some oxygen deficit which causes an initial DO deficit in the stream
- Water can only hold so much oxygen ($DO_{sat}$), depending on the the water temperature
- Calculate the initial dissolved oxygen ($DO_0$) using the same formula we used for $L_0$ above
- Subtracting that from the initial $DO_{sat}$:

$$D_0 = DO_{sat} - DO_0$$

The DO at any point downstream depends on these competing processes:

rate of deficit increase = rate of deoxygenation - rate of reaeration

- This gives a differential equation with the solution:

$$D = \frac{k_D L_0}{k_R - k_D}\left(e^{-k_D t} - e^{-k_R t}\right) + D_0 e^{-k_R t}$$

- This is the Streeter-Phelps oxygen-sag curve formula
- Note that for a constant stream cross-section, t=x/u (with u=stream velocity); therefore:

$$D = \frac{k_D L_0}{k_R - k_D}\left(e^{-k_D x/u} - e^{-k_R x/u}\right) + D_0 e^{-k_R x/u}$$

- to plot DO versus distance downstream we need to subtract D from $D_s$ at each point

To start with, DO is being depleted faster than it can be replenished

- As long as this occurs, the DO of the stream will continue to drop
- Since the BOD is decreasing as time goes on, at some point, the rate of deoxygenation decreases to just the rate of reaearation
    - At this point (called the critical point) the DO reaches a minimum
    - Downstream of the critical point, reaeration occurs faster than deoxygenation, so the DO increases
- Using calculus and the Streeter-Phelps equation we get:

  Critical Time at which Max. DO deficit occurs , $t_c$

$$t_c = \frac{1}{k_R - k_D} \ln\left(\frac{k_R}{k_D}\left[1 - \frac{D_0(k_R - k_D)}{k_D L_0}\right]\right)$$

**Problem:**

100 cumece of sewage of a city is discharged in a perennial river which is fully saturated with oxygen and flows at a minimum rate of 1250 cumecs with a minimum velocity of 0.15 m/sec. If the 5-day BOD of the sewage is 260 mg/l, find out where the critical DO will occur in the river. Assume :

(i) The coefficient of purification of river as 4.0

(ii) Coefficient of DO as 0.11

and (iii) The ultimate BOD as 125% of the 5 day BOD of the mixture of sewage and river water.

---

**Answer:**

Let us assume a temperature of 20° C, for which saturation DO is equal to 9.17 mg/l. Also, assume that DO of effluent is zero.

$$\therefore \qquad (DO)_{mix} = \frac{(1250 \times 9.17) + (100 \times 0)}{1250 + 100} = 8.49 \text{ mg/l.}$$

Hence initial DO deficit $= D_0 = 9.17 - 8.49 = 0.68$ mg/l.

Also, 5-day BOD of sewage $= 260$ mg/l.

Let 5-day BOD of river $= 0$

$$\therefore \qquad (y_5)_{mix} = \frac{(1250 \times 0) + (100 \times 260)}{1250 + 100} = 19.26 \text{ mg/l.}$$

It is given that ultimate BOD is equal to 125% of the 5-day BOD of the mixture of sewage and river water.

$$\therefore \qquad L_0 = 1.25\,(y_5)_{mix} = 1.25 \times 19.26 = 24.07 \text{ mg/l.}$$

Thus, both $D_0$ and $L_0$ are known. Now, from Eq.

---

$$t_c = \frac{1}{K(f_s - 1)} \log_{10}\left[f_s\left\{1 - (f_s - 1)\frac{D_0}{L_0}\right\}\right]$$

Here, $K = 0.11$ (given ) and $f_s = 4.0$ (given)

$$\therefore \qquad t_c = \frac{1}{0.11\,(4-1)} \log_{10}\left[4\left\{1 - (4-1)\frac{0.68}{24.07}\right\}\right] \doteq 1.7078$$

$$\therefore \qquad x_c = \text{velocity} \times \text{time} = 0.15\,(1.7078 \times 24 \times 3600) \times 10^{-3}$$
$$= \mathbf{22.13\ km}$$

Hence critical deficit will occur at 22.13 km downstream of the sewage disposal point.

---

**Problem:**

**A town discharges 100 m$^3$/sec of sewage into a stream having a rate of flow of 1200 m$^3$/sec during lean days, at a 5 day BOD of sewage at the given temperature being 250 mg/l. Find the amount of critical D.O deficit and its location in the downstream portion of the stream. Assume de-oxygenation co-efficient K as 0.1 and co- efficient of self purification as 3.5. Take saturation dissolved oxygen at given temperature as 9.2 mg/l.**

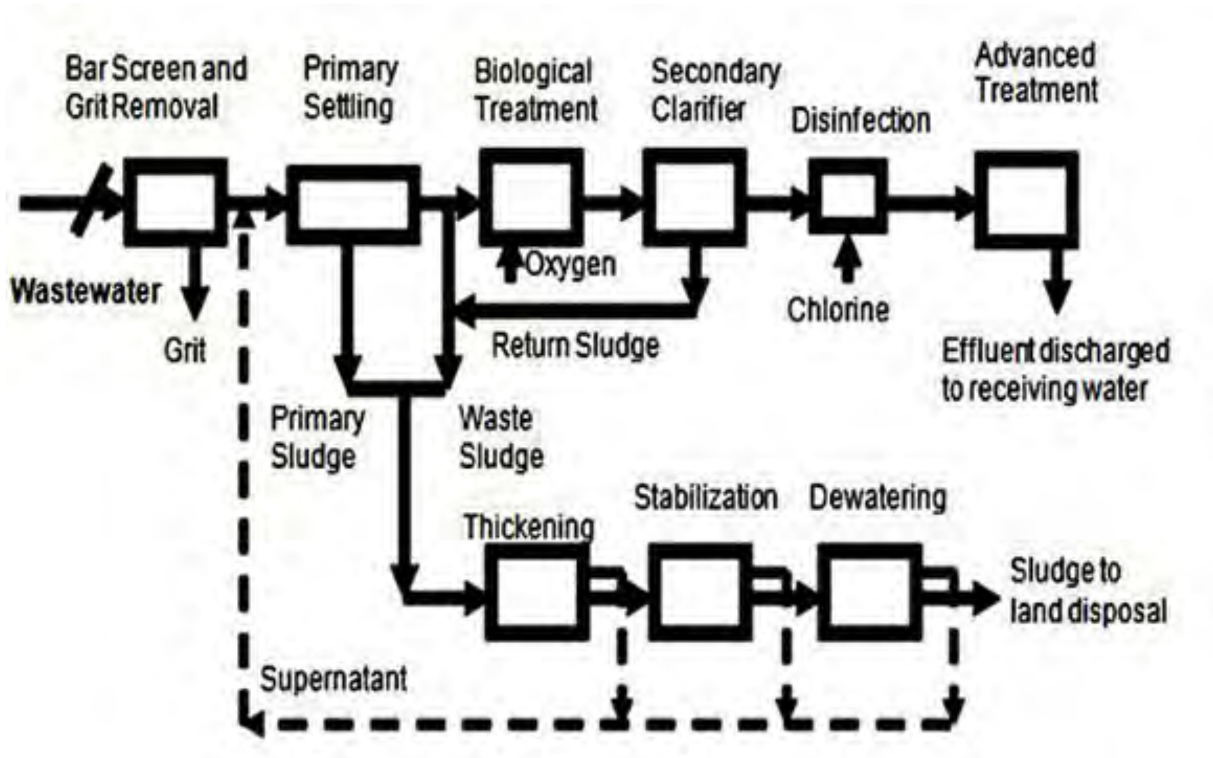
**Answer:**

**$L_{(20)}$ = 365.62 mg/l**
**$C_{BOD}$ = 15.625 mg/l**
**Amount of critical D.O deficit, $D_c$ = 0.78 mg/l**
**Time, $t_c$ = 2.07 day**
**Distance down stream = 17.93 km**

# Unit-V

## TREATMENT OF WASTE WATER

### Flow diagram of Municipal waste water treatment plant



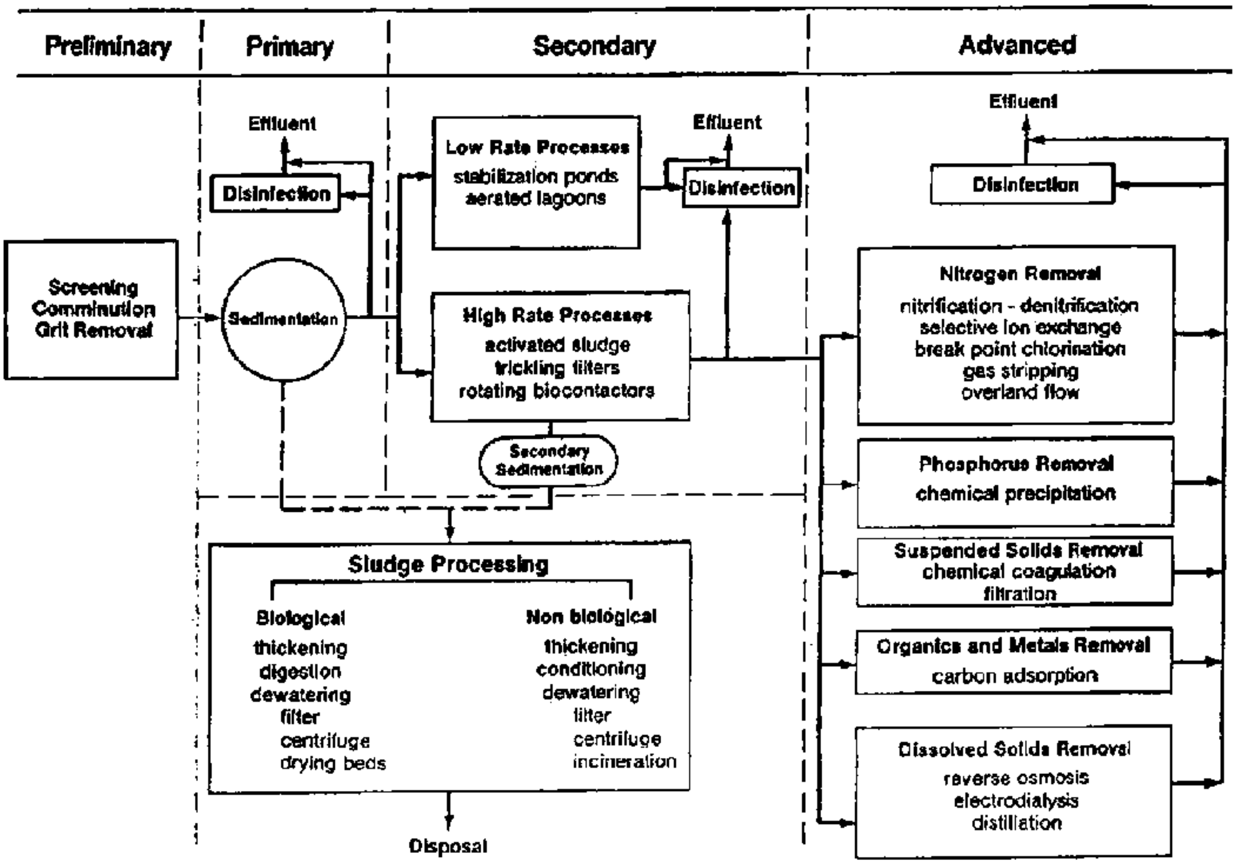

The principal objective of wastewater treatment is generally to allow human and industrial effluents to be disposed of without danger to human health or unacceptable damage to the natural environment. Irrigation with wastewater is both disposal and utilization and indeed is an effective form of wastewater disposal (as in slow-rate land treatment). However, some degree of treatment must normally be provided to raw municipal wastewater before it can be used for agricultural or landscape irrigation or for aquaculture. The quality of treated effluent used in agriculture has a great influence on the operation and performance of the wastewater-soil-plant or aquaculture system. In the case of irrigation, the required quality of effluent will depend on the crop or crops to be irrigated, the soil conditions and the system of effluent distribution adopted. Through crop restriction and selection of irrigation systems which minimize health risk, the degree of pre-application wastewater treatment can be reduced. A similar approach is not feasible in aquaculture systems and more reliance will have to be placed on control through wastewater treatment.

The most appropriate wastewater treatment to be applied before effluent use in agriculture is that which will produce an effluent meeting the recommended microbiological and chemical quality guidelines both at low cost and with minimal operational and maintenance requirements. Adopting as low a level of treatment as possible is especially desirable in developing countries, not only from the point of view of cost but also in acknowledgement of the difficulty of operating complex systems reliably. In many locations it will be better to design the reuse system to accept a low-grade of effluent rather than to rely on advanced treatment processes producing a reclaimed effluent which continuously meets a stringent quality standard.

Nevertheless, there are locations where a higher-grade effluent will be necessary and it is essential that information on the performance of a wide range of wastewater treatment technology should be available. The design of wastewater treatment plants is usually based on the need to reduce organic and suspended solids loads to limit pollution of the environment. Pathogen removal has very rarely been considered an objective but, for reuse of effluents in agriculture, this must now be of primary concern and processes should be selected and designed accordingly. Treatment to remove wastewater constituents that may be toxic or harmful to crops, aquatic plants (macrophytes) and fish is technically possible but is not normally economically feasible. Unfortunately, few performance data on wastewater treatment plants in developing countries are available and even then they do not normally include effluent quality parameters of importance in agricultural use.

The short-term variations in wastewater flows observed at municipal wastewater treatment plants follow a diurnal pattern. Flow is typically low during the early morning hours, when water consumption is lowest and when the base flow consists of infiltration-inflow and small quantities of sanitary wastewater. A first peak of flow generally occurs in the late morning, when wastewater from the peak morning water use reaches the treatment plant, and a second peak flow usually occurs in the evening. The relative magnitude of the peaks and the times at which they occur vary from country to country and with the size of the community and the length of the sewers. Small communities with small sewer systems have a much higher ratio of peak flow to average flow than do large communities. Although the magnitude of peaks is attenuated as wastewater passes through a treatment plant, the daily variations in flow from a municipal treatment plant make it impracticable, in most cases, to irrigate with effluent directly from the treatment plant. Some form of flow equalization or short-term storage of treated effluent is necessary to provide a relatively constant supply of reclaimed water for efficient irrigation, although additional benefits result from storage.

**Conventional wastewater treatment processes**




## Preliminary treatment

The objective of preliminary treatment is the removal of coarse solids and other large materials often found in raw wastewater. Removal of these materials is necessary to enhance the operation and maintenance of subsequent treatment units. Preliminary treatment operations typically include coarse screening, grit removal and, in some cases, comminution of large objects. In grit chambers, the velocity of the water through the chamber is maintained sufficiently high, or air is used, so as to prevent the settling of most organic solids. Grit removal is not included as a preliminary treatment step in most small wastewater treatment plants. Comminutors are sometimes adopted to supplement coarse screening and serve to reduce the size of large particles so that they will be removed in the form of a sludge in subsequent treatment processes. Flow measurement devices, often standing-wave flumes, are always included at the preliminary treatment stage.

The objective of primary treatment is the removal of settleable organic and inorganic solids by sedimentation, and the removal of materials that will float (scum) by skimming. Approximately 25 to 50% of the incoming biochemical oxygen demand ($BOD_5$), 50 to 70% of the total suspended solids (SS), and 65% of the oil and grease are removed during primary treatment. Some organic nitrogen, organic phosphorus, and heavy metals associated with solids are also removed during primary sedimentation but colloidal and dissolved constituents are not affected. The effluent from primary sedimentation units is referred to as primary effluent.

In many industrialized countries, primary treatment is the minimum level of pre application treatment required for wastewater irrigation. It may be considered sufficient treatment if the wastewater is used to irrigate crops that are not consumed by humans or to irrigate orchards, vineyards, and some processed food crops. However, to prevent potential nuisance conditions in storage or flow-equalizing reservoirs, some form of secondary treatment is normally required in these countries, even in the case of non-food crop irrigation. It may be possible to use at least a portion of primary effluent for irrigation if off-line storage is provided.

Primary sedimentation tanks or clarifiers may be round or rectangular basins, typically 3 to 5 m deep, with hydraulic retention time between 2 and 3 hours. Settled solids (primary sludge) are normally removed from the bottom of tanks by sludge rakes that scrape the sludge to a central well from which it is pumped to sludge processing units. Scum is swept across the tank surface by water jets or mechanical means from which it is also pumped to sludge processing units.

In large sewage treatment plants (> 7600 $m^3/d$ in the US), primary sludge is most commonly processed biologically by anaerobic digestion. In the digestion process, anaerobic and facultative bacteria metabolize the organic material in sludge (see Example 3), thereby reducing the volume requiring ultimate disposal, making the sludge stable (non-putrescible) and improving its dewatering characteristics. Digestion is carried out in covered tanks (anaerobic digesters), typically 7 to 14 m deep. The residence time in a digester may vary from a minimum of about 10 days for high-rate digesters (well-mixed and heated) to 60 days or more in standard-rate digesters. Gas containing about 60 to 65% methane is produced during digestion and can be recovered as an energy source. In small sewage treatment plants, sludge is processed in a variety of ways including: aerobic digestion, storage in sludge lagoons, direct application to sludge drying beds, in-process storage (as in stabilization ponds), and land application.

**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. In most cases, secondary treatment follows primary treatment and involves the removal of biodegradable dissolved and colloidal organic matter using aerobic biological treatment processes. Aerobic biological treatment (see Box) 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 $CO_2$, $NH_3$, and $H_2O$). 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.

High-rate biological processes are characterized by relatively small reactor volumes and high concentrations of microorganisms compared with low rate processes. Consequently, the growth rate of new organisms is much greater in high-rate systems because of the well controlled environment. The microorganisms must be separated from the treated wastewater by sedimentation to produce clarified secondary effluent. The sedimentation tanks used in secondary treatment, often referred to as secondary clarifiers, operate in the same basic manner as the primary clarifiers described previously. The biological solids removed during secondary sedimentation, called secondary or biological sludge, are normally combined with primary sludge for sludge processing.

Common high-rate processes include the activated sludge processes, trickling filters or bio-filters, oxidation ditches, and rotating biological contactors (RBC). A combination of two of these processes in series (e.g., bio-filter followed by activated sludge) is sometimes used to treat municipal wastewater containing a high concentration of organic material from industrial sources.

**i. *Activated Sludge***

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. Hydraulic retention time in the aeration tanks usually ranges from 3 to 8 hours but can be higher with high $BOD_5$ wastewaters. 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. Several variations of the basic activated sludge process, such as extended aeration and oxidation ditches, are in common use, but the principles are similar.

**ii. *Trickling Filters***

A trickling filter or bio-filter 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. 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. Forced air can also be supplied by blowers but this is rarely necessary. 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 bio-filter to improve hydraulic distribution of the wastewater over the filter.

**iii. *Rotating Biological Contactors***

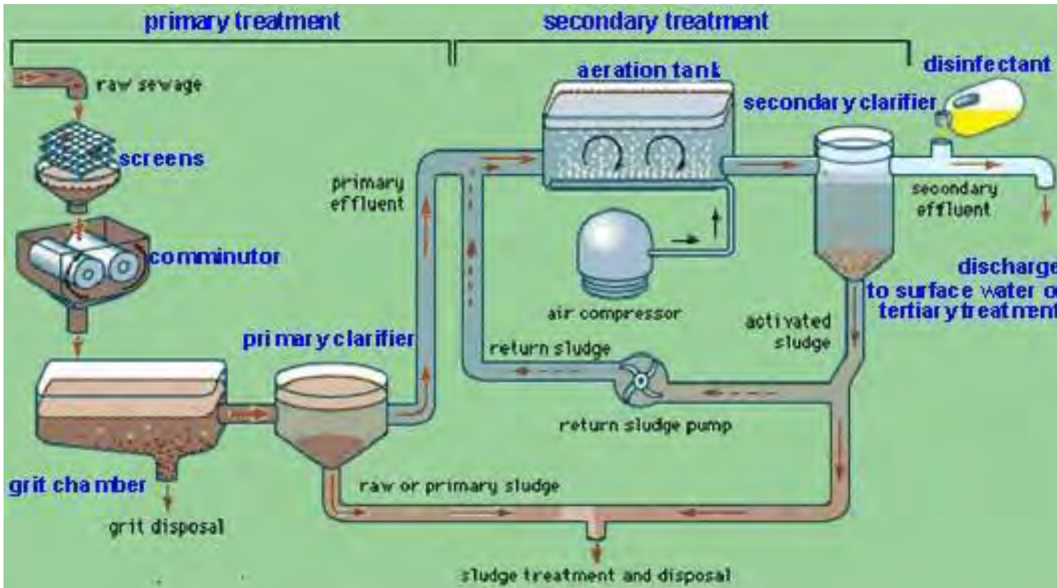
Rotating biological contactors (RBCs) are fixed-film reactors similar to bio-filters in that organisms are attached to support media. In the case of the RBC, the support media are slowly rotating discs that are partially submerged in flowing wastewater in the reactor. Oxygen is supplied to the attached bio-film from the air when the film is out of the water and from the liquid when submerged, since oxygen is transferred to the wastewater by surface turbulence created by the discs' rotation. Sloughed pieces of bio-film are removed in the same manner described for bio-filters.

High-rate biological treatment processes, in combination with primary sedimentation, typically remove 85 % of the $BOD_5$ and SS originally present in the raw wastewater and some of the heavy metals. Activated sludge generally produces an effluent of slightly higher quality, in terms of these constituents, than bio-filters or RBCs. When coupled with a disinfection step, these processes can provide substantial but not complete removal of bacteria and virus. However, they remove very little phosphorus, nitrogen, non-biodegradable organics, or dissolved minerals.

**Tertiary and/or advanced treatment**

Tertiary and/or advanced wastewater treatment is employed when specific wastewater constituents which cannot be removed by secondary treatment must be removed. As shown in Figure , individual treatment processes are necessary to remove nitrogen, phosphorus, additional suspended solids, refractory organics, heavy metals and dissolved solids. Because advanced treatment usually follows high-rate secondary treatment, it is sometimes referred to as tertiary treatment. However, advanced treatment processes are sometimes combined with primary or secondary treatment (e.g., chemical addition to primary clarifiers or aeration basins to remove phosphorus) or used in place of secondary treatment (e.g., overland flow treatment of primary effluent).

**Screening and Grit chambers**

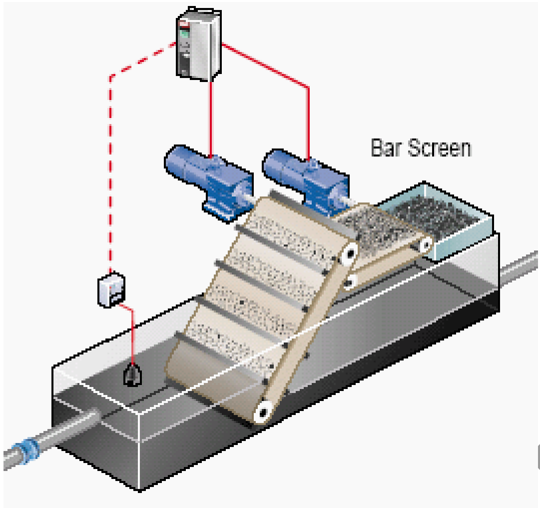



The removal of solids from the incoming wastewater flow is accomplished in steps. The first steps involve physical separation of the solids by screening and by gravity. The larger solids can be removed using screens and the heavy solids can be removed using settling processes. The dissolved organic material (and some of the lighter suspended solids) will remain in the sewage flow after primary clarification. The first step in the solids removal process is screening to remove the larger solids and "rags." After screening, a grit removal process is used to separate the heavier inorganic solids like sand and inert organics like coffee grounds from the flow. Rags can clog piping and pumps in downstream processes. Grit can also cause clogging problems and can damage pumps. Grit that isn't removed in the grit chamber will end up in the solids handling system where it will eventually collect in the digesters. This will reduce available digester capacity. These processes that remove inorganic solids are collectively referred to as pretreatment.

### BAR SCREENS

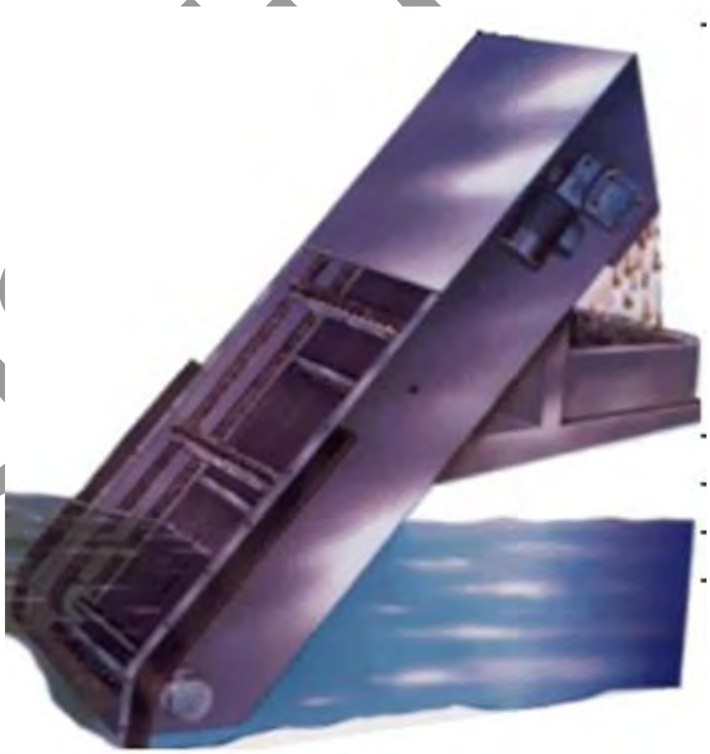
Bigger pieces of wood, plastic, metal, rubber, textile and other waste materials out of wastewater is being removed by **using bar screens**.

A bar screen consists of a series parallel of steel bars that are placed vertically in the influent flow channel. The bars are usually spaced about 1/2-3/4 inches apart. In some cases, two sets of screens are placed in the channel the upstream screen may have bars spaced 2-3 inches apart and the downstream screen will have the normal spacing. The front screen is sometimes referred to as a "trash screen." It is designed to catch large chunks of debris to avoid overloading the smaller screen. As the screen gets clogged with rags the water level upstream will rise. If the screen isn't cleaned regularly the upstream water level can back up and flood the structure.




**Automatically cleaned bar screens**

An operator must rake the collected debris from a manually cleaned bar screen. Manually cleaned bar screens are usually set at a 45-degree angle. This makes it easier to rake the debris from the screen.



**Manually cleaned bar screens**

Automatically cleaned bar screens are designed with a set of rakes that are chain-driven. These units will operate periodically to remove the rags and deposit them in some type of container. The bar screen angle is usually between 60 and 90 degrees on an automatically cleaned screen system. The rags that are removed by the screen must be hauled to a landfill for disposal. However, some screen systems actually rake the screens, grind or shred the rags, and then return them to the waste flow.

Incoming septic sewage can cause corrosion problems with steel screens. Hydrogen sulfide formed by anaerobic decomposition will attack the metal bars. Bar screens should be inspected several times a year for corrosion and bent bars. Repair and replacement of the bars is the only maintenance issue for manually cleaned screens. Automatically cleaned screens need to have weekly inspections to check the conditions of the rake teeth and the chain drive.

**Bar Screens are used to separate large debris such as rags and plastics.**

**Two types of bar screens:**

1. Coarse sieve (***rake***) and
2. Fine sieve.

***Coarse sieves*** have openings equal to or greater than 6 mm and generally protective role, while using a ***fine sieve***, with openings smaller than 6 mm, and can achieve significant removal of suspended solids from the wastewater.

The grid is usually made up of ***parallel rods***, while the fine screen usually used wire cloth or perforated metal plate.

## COMMINUTORS AND GRINDER PUMPS

A comminutor is a device that is designed to shred rags and debris into small pieces. It takes the place of a bar screen. Debris collects on the cage of the comminutor and a set of revolving teeth cut the rags up into pieces small enough that they won't clog pumps and pipes. Grinder pumps or macerators, such as the "Muffin Monster," also grind up debris as it flows through the pump. Comminutors or grinders will normally be placed in parallel with a manually cleaned bar screen. The bar screen is usually set up in parallel so it can be used when the comminutor is down for service.

**Grit Chambers**

Grit chambers are basin to remove the inorganic particles to prevent damage to the pumps, and to prevent their accumulation in sludge digestors.

A portion of the suspended solids load of municipal waste water consists of grit material. If not removed in preliminary treatment, grit in primary settling tank can cause abnormal abrasive wear on mechanical equipment and sludge pumps, can clog by deposition, and can accumulate in sludge holding tanks and digesters. Therefore grit removal is necessary to protect the moving mechanical equipments and pump elements from abrasion and accompanying wear and tear. Removal of grit also reduces the frequency of cleaning of digesters and settling tanks.

Grit removal devices rely upon the difference in specific gravity between organic and inorganic solids to effect their separation.

Composition of grit

Grit in sewage consists of coarse particles of sand, ash and clinkers, egg shells, bone chips and many inert materials inorganic in nature. Both the quantity and quality of grit varies depending upon

a) Types of street surfaces encountered
b) Relative areas served
c) Climatic conditions

d) Types of inlet and catch basins
e) Amount of storm water diverted
f) Sewer grades
g) Ground water characteristics, etc.,
h) Social habits

The specific gravity of grit is usually in the range of 2.4 to 2.65. Grit is non putrescible in nature and higher hydraulic subsidence than organic solids. Hence it is possible to separate the gritty material from organic solids by differential sedimentation in a grit chamber.

Types of grit chambers

Grit chambers are of two types, mechanically cleaned and manually cleaned. The choice depends on several factors such as the quantity and quality of grit to be handled, head loss requirements, space requirements etc.,

Mechanically cleaned grit chambers are provided with mechanical equipment for collection, elevation and washing of grit which are operated either on a continuous or intermittent basis.

Manually cleaned grit chambers should have sufficient capacity for storage of grit between the intervals of cleansing. Atleast two tanks must be there, so that when one is under cleaning operation the other must be working. These tanks must be cleaned atleast once in every week. The simplest method of removal is by means of shovel and wheel barriers.

Aerated grit chambers

An aerated grit chamber is a special form of grit chamber consisting of a standard spiral flow aeration tank provided with air diffusion tubes placed on one side of the tank, 0.6 to 1m from the bottom. The heavier grit particles with their higher settling velocities drop down to the floor where as the lighter organic particles are remained in suspension and carried with the roll of the spiral motion, due to the diffused air and eventually out of the tank.

**Principle of Working of Grit Chamber**

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. Hence, the flow velocity should neither be too low as to cause the settling of lighter organic matter, nor should it be too high as not to cause the settlement of the silt and grit present in the sewage. This velocity is called "differential sedimentation and differential scouring velocity". The scouring velocity determines the optimum ***flow through velocity***. This may be explained by the fact that the critical velocity of flow '$v_c$' beyond which particles of a certain size and density once settled, may be again introduced into the stream of flow. It should always be less than the scouring velocity of grit particles. The critical velocity of scour is given by Schield's formula:

$V = 3 \text{ to } 4.5\,(g(S_s - 1)d)^{1/2}$

A horizontal velocity of flow of 15 to 30 cm/sec is used at peak flows. This same velocity is to be maintained at all fluctuation of flow to ensure that only organic solids and not the grit is scoured from the bottom.

**Types of Velocity Control Devices**

1. A sutro weir in a channel of rectangular cross section, with free fall downstream of the channel.
2. A parabolic shaped channel with a rectangular weir.

3. A rectangular shaped channel with a parshall flume at the end which would also help easy flow measurement.

Design data

The basic data essential for a rational approach to the design of grit chambers are hourly variations of sewage flow and typical values for minimum, average and peak flows. Since the grit chamber is designed for peak flows and the flow through velocity is maintained constant within the range of flow, successful design and operation of grit chamber calls for a fairly accurate estimate of flow.

The quality and quantity of grit varies from sewage to sewage. Data relating to these two factors is very useful in proper design of grit collecting, elevating and washing mechanisms. In the absence of specific data, grit content may be taken as 0.025 to 0.075m$^3$ /million litres for domestic sewage and 0.06 to 0.12m$^3$/ million litres for combined sewage.

Design of grit chambers

i. Settling velocity or hydraulic subsidence value:

The settling velocity is governed by the size and specific gravity of grit particles and the viscosity of the sewage. The size of separation based on the minimum size and of grit to be removed is 0.2mm. The sp.gr of the grit may be as low as 2.4, but for design purposes a value of 2.65 considered as grit. The settling velocity is given by

$$Vs = 60.6(\delta s - \delta)\frac{3T+70}{100}$$

$$= 60.6(Ss - 1)\, d\, \frac{3T+70}{100}$$

Where,

$Vs$= settling velocity in cm/sec

$Ss$= $\delta s/\delta$= specific gravity

$\delta s$= mass density of particle in gm/cc

δ= mass density of liquid in gm/cc

d= particle size in cm

T= temperature of liquid in $^o$c

For sp.gr of grit equal 2.65 and liquid equal to 1,

$Vs = 60.6(2.65 - 1)\ d\ \frac{3T+70}{100} = d\ (3T + 70)$

For organic solids whose sp.gr equal to 1.2,

$V_S = 0.12\ d\ (3T + 70)$

Settling velocity is also called as Hydraulic subsidence rate (H.S.V)

ii. Overflow rate

A grit chamber designed for removal of 100% of grit particles of smallest size would also remove all grit particles larger than this. To obtain a 100% removal of the smallest size particles, it would be theoretically necessary for the detention period in the tank equal to the time required for the minimum sized particles to reach the tank bottom. In other words the conditions should be ideal for settling velocity particles. It can also be shown that the settling velocity $V_S$ of the minimum sized particles is equal to surface loading rate (Q/A) or overflow rate in order to obtain a theoretical 100% removal of the particles.

The overflow rate of 1300 to 1700 $m^3/m^2$/day be taken for particles of 0.15 to 0.2mm dia and sp.gr 2.65 at $10^oc$, which may be converted to any other temperature by the factor

$$\frac{3T + 70}{100}$$

iii. Flow through velocity

A horizontal velocity or flow through velocity of 15cm/sec to 30cm/sec is used at peak flows.

iv. Detention period

A period of 60 secs or one minute is usually adopted.

Detritus tank

A detritus tank is similar to a grit chamber. The difference being only in the velocity of flow and the detention period. A detritus tank may be considered as a grit chamber in which the velocity of flow is such that an appreciable amount of organic matter settles down along matter by blowing the compressed air through the detritus tanks in order to lift the lighter organic solids or by washing in a grit washer.

Design aspects of detritus tank

Detritus tanks are normally rectangular in shape. The sides are vertical but tapered at bottom to form a through for the detritus collection. Overall depth of detritus tank may vary from 2.5 to 3.5m and detention period 3 to 4 minutes. Velocity of flow is kept between 20cm/sec to 40cm/sec.

Velocity control devices

In order to maintain a constant horizontal velocity of flow through grit chamber in the recommended range of 15 to 30 cm/sec, irrespective of flows, devices like proportional weir, sutro weir, parshal flume etc, have been designed.

**Problem**

Design a grit chamber for a town having a population of 1 lakh. Assume suitable data necessary.

Solution:

Assuming per capita sewage production as 120 litres/day

Average daily flow = 1, 00,000 X 120 litres

$$= \frac{12X10^6}{1000X24X60X60}$$

$$= 0.139 \text{ m}^3/\text{sec}$$

Maximum flow = 3X 0.139 = 0.417 $m^3$/sec

Assuming a horizontal velocity of 30 cm/sec, and detention time as 60 seconds,

Cross section area of the tank = A = $\frac{FLOW}{VELOCITY} = \frac{0.417}{0.3}$ = 1.39 $m^2$

Length of the tank required = L = horizontal velocity of flow X detention time

= 0.3 X 60 = 18 m.

There fore capacity of the tank = c/s area X length = 1.39 X 18

V = 25 $m^3$

Quantity of grit at the rate of 25 litres/million litres/day = $\frac{120X10^5X25}{10^6}$ = 300litres/day

Assuming cleaning of grit chamber is once in a week,

Storage for one week = 7 X 300 = 2100 litres = 2.1 $m^3$

Total tank capacity = 25 + 2.1 = 27.1 $m^3$

Assuming the size of the particles as 0.2 mm and sp.gr 2.65, and temperature as 20 $^O$c,

Settling velocity given by Hazen formula,

$V_S$ = d (3T + 70) = 0.02 (3 X 20 + 70) = 2.2 cm/sec.

Therefore depth of the tank required = settling velocity X detention time = 2.2 X 60 = 132 cm

Liquid depth = 1.32m

Breadth = $\frac{1.39\text{m}^2}{1.32\text{m}}$ = 1.05 m

Provide 0.27 m free board.

Total depth of tank = 1.32 + 0.27 + 0.11 m sludge depth

Depth = 1.7 m; Breadth = 1.05 m; Length = 18 m

Provide two chambers of above dimensions so that when one is in cleaning operation, the other will be working.

Oil and Grease removal

Grease in sewage include fats, waxes, free fatty solids, calcium and magnesium soaps, mineral oils, etc., oil and grease find their way in sewage from restaurants, kitchens, garages, soap and candle factories, oil refineries, slaughter houses etc.,. If not removed, these substances may create the following difficulties:

1) If sewage is being discharged into the stream for disposal, unsightly scum, and foul odour may be developed at the surface of the stream. The scum retards reoxygenation and thus causes anaerobic condition.
2) They do not digest easily and therefore create problems in sludge digestion tanks.
3) They interfere with some of the treatment processes and also promote clogging of the trickling filters.
4) They affect the biological activities of the organisms and thus affect their smooth working.

They can be removed from sewage either by floatation or settling as scum or sludge. Formation of scum is promoted by diffusing air through the sewage. The tank in which scum formation is promoted by diffusion of air through the sewage is called skimming tank.

Skimming tanks

Skimming tanks are narrow rectangular tanks having at least two longitudinal baffle walls, interconnected. They are used to remove grease and fatty oils from the sewage. Air diffusers are provided at the bottom of the tank. Compressed air applied at the rate varying from 300 to 6000 $m^3$ per million litres of sewage agitates the sewage, which prevents settling of solids. Air tends to change the oil and grease to a soapy mixture. This mixture is carried to the surface by the air bubbles, some of which are entrained in it and may be skimmed off.

Design aspects

The ratio of length to depth of skimming tank should be about 2 or 1.5 to 1. Usual detention period is 3 minutes.

The surface area required for the tank can be found by the formula

$$A = 1110q / v_r$$

Where,

A = surface area in sq.ft

q = rate of sewage flow in million gallons / day

$V_r$ = minimum rising velocity of oily materials to be removed in inches/minute. Value of $V_r$ in most of the cases is 25 cm/min.

## PRIMARY TREATMENT – SEDIMENTATION

In preliminary treatment, removal of coarse natural solids, debris, grit, oil and grease etc., takes place. Finer suspended solids that cannot be removed from sewage from these processes can be removed by the process known as sedimentation. The process of sedimentation is carried out in tanks known as settling tanks or sedimentation tanks or clarifiers.

Usually at sewage treatment plants, this sedimentation is carried twice, once before the biological treatment and once after the biological treatment. Sedimentation before the biological treatment is called primary settling and that carried after the biological treatment is called secondary settling. If settleable solids are separated from sewage by gravitation and by natural flocculation alone, the process is termed as plain sedimentation. If chemicals are used to induce, or increase flocculation, the process is called chemical precipitation.

Sedimentation is the separation from water, by gravitation settling, of suspended particles that are heavier than water. It is one of the most widely used unit operations in the waste water treatment. The principles on which design of sedimentation tanks depends are that when a liquid containing solids in suspension is placed in a relatively quiescent state, those solids having a higher specific gravity than the liquid will tend to settle, and those with a lower specific gravity will tend to rise.

The well known Stokes‘s law on sedimentation expresses the relationship between the settling velocity size and density of particles settled, density and viscosity of liquid is given by

$$Vs = \frac{g\, d2\,(\delta s - \delta)}{18\mu} \quad \text{or} \quad \left(\frac{g}{18}\right)\left(\frac{Ss - 1}{\mu}\right) d$$

(For particles of size less than 0.1 mm)

And Newton‘s for the same is

$V_S = 1.8\ g\ (S_S - 1)\ d$

(For particles of size more than 1mm)

Where,

$Vs$ = settling velocity of discrete particular in cm/sec

g = gravitational constant in cm/sec$^2$

μ = absolute or dynamic viscosity of the fluid in centi-poise

$\delta s$ = mass density of particles in gm/cc

$\delta$ = mass density of liquid in gm/cc

Ss = $\delta s$ / $\delta$ = Sp.gr.

(1 centi poise = $10^{-2}$ poise or $10^{-2}$ dynes)

(1 centi stoke = $10^{-2}$ stokes or $10^{-2}$ $cm^2$/sec)

For the particular of size between 0.1mm, Hazen modification formula is

$Vs = d\,(3T + 70)$ ,

Where,

T = temperature of liquid, which applied for grit particles of sp.gr. 2.65

Characteristics of settleable solids

The settleable solids to be removed in sedimentation tanks are mainly organic in nature, dispersed or flocculated. The specific gravity of these suspended solids may vary from 1.01 to 1.20. Generally raw sewage is a dilute heterogeneous suspension of low specific gravity ranging from fully dispersed to completely flocculated ones.

Process of sedimentation

In the process of sedimentation the velocity of flow is decreased to such a value that finer settleable will settle to the bottom of the tank. Such settling tanks in which sewage continuously keeps on moving with predetermined velocity are called continuous flow settling tanks. In olden days sewage was used to be filled in large tanks and allowed to remain quiescent for some time and then they were emptied. Such tanks are called fill and draw type tanks.

The liquid sewage coming out of settling tanks after the process of sedimentation is called effluent and the settled viscous sewage at the bottom of tank is called sewage sludge.

Design considerations

Several factors such as flow variations, density currents, solids concentrations, solids loading, area, detention time and overflow rate influence the design and performance of sedimentation tanks. Sedimentation tanks are designed for average flow condition.

Overflow rate or surface loading rate

The overflow rate represents the hydraulic loading per unit surface area of tank in unit time expressed as $m^3/m^2$/day. Overflow rates must be checked both at average and peak flows.

Detention period

The rate of removal of BOD and SS is maximum during the first 2 to 2.5 hours of settling and thereafter decreases appreciably. Longer detention beyond 4 hours may affect the tank performance adversely due to settling in septic conditions, particularly in tropical countries. Experience has shown that a detention period of 2 to 2.5 hours for primary settling tanks and 1.5 to 2 hours for secondary settling tanks would produce the optimum results.

Weir loading

Weir loading influences the removal of solids in particularly secondary settling tanks. It has been found that weir rates have less influence on the efficiency of settling tanks.

Depth

The depth sets the detention time in the settling tanks and also influences the sludge thickening in secondary settling tanks.

Sludge removal

Sludge can be removed manually, hydrostatically or mechanically from the sedimentation tanks. Mechanical cleaning of sludge should be preferred to manual cleaning even in small plants, where power is available for running the plant machinery. Even when power is not available or inadequate, hydrostatic removal should be adopted to avoid manual handling of sludge to prevent exposure of workers to health hazards.

Inlets and outlets

Performance of sedimentation tank is very much influenced by inlet devices which are intended to distribute and draw the flow evenly across the basin. All the inlets must be designed to keep the entrance velocity to prevent the formation of eddy or inertial currents in the tank to avoid short circuiting.

In horizontal flow rectangular tanks, inlets and outlets are placed opposite to each other separated by the length of tank with the inlet perpendicular to direction of flow. Inlet may be multiple pipe type, channel inlet with baffles or an inlet channel with submerged weir.

Outlet is generally an overflow weir located near the effluent end, preferably adjustable for maintaining the weir at a constant level. V- notches are provided on the weir to provide for uniform distribution of flow at low heads of discharge over the weir.

Types and shapes of sedimentation tanks

Circular tanks are more common than rectangular or square tanks. Upflow tanks have been used for sewage sedimentation but horizontal flow types are more popular. Rectangular tanks need less space than circular tanks and could be more economically designed where multiple units are required to be constructed.

For rectangular tanks, maximum length and widths may be 90m and 30m respectively with length to width ratios of 1.5 to 7.5 and length to depth ratios of 5 to 25 are recommended. A minimum of 2m in primary settling and 2.5m in case of secondary settling tanks are provided.

Diameters of circular tanks vary widely from 3 to 60m although the most common range is 12 to 30m. The water depth varies from 2m for primary to 3.5m for secondary settling tanks. Floors are sloped from periphery to centre at a rate of 7.5 to 10%. The inlet is generally at centre and outlet is a peripherical weir, the flow being radial and horizontal from centre to the periphery of the tank.

Performance

Primary sedimentation of domestic sewage may be expected to accomplish 30 to 45% removal of BOD and 45 to 60% removal of SS depending on the concentration and characteristics of solids in suspension. Secondary settling tanks if considered independently, remove a very high percentage of flocculated solids, even more than 99 %, under certain situations.

Table may be used for design of sedimentation tanks.

| Sl. No | Item | Over flow rate $m^3/m^2/day$ | Detention time hours | Weir loading $m^3/m/day$ | Depth m | Dimensions m |
|---|---|---|---|---|---|---|
| 1. | Primary settling only | 30-60 | 2-2.5 | 100-150 | 3-3.5 | Length=25-40 Width=6-10 |
| 2. | Primary settling followed by secondary treatment | 30-100 | 1.5-2 | 150-300 | 3-3.5 | Depth=3.6 Circular |
| 3. | Primary settling with activated sludge return | 30-60 | 1.5-2.5 | 150-300 | 3.5-4.0 | dia.=12-45m Depth=4-5m |
| 4. | Secondary settling for trickling filter | 15-45 | 2-2.5 | - | 3.4 | |
| 5. | Secondary settling for activated sludge return | 15-50 | 2-2.5 | - | 3.5-4.5 | |
| 6. | Secondary settling for extended aeration | 10-40 | 3-4 | - | 3.5-5.0 | |

**Problem**

Design primary settling tanks required to treat the sewage from a town of population one lakh.

**Answer:**

Assuming per capita production of sewage as 120 litres/day

Average daily sewage flow = 120 X $10^5$ litres

$$= \frac{120 \text{ X } 10^6}{1000 \text{ X } 24} = 500 \text{ m}^3\text{/hour}$$

Assuming a detention period of 2 hours,

Capacity of the tank = 500 X 2 = 1000 $m^3$

Assuming a depth of 3 m, the surface area required = 1000/3 = 333.33 $m^2$

Diameter of the tank:

$$\frac{\pi d^2}{4} = 333.33\, m^2$$

Therefore d= $\frac{\sqrt{4 \ X\ 333.33}}{\pi}$

d = 20.6 m say 21m

Check:

Overflow rate = Q/A = $\frac{12\ X\ 10^6\ X\ 4}{1000\ X\ \pi\ X\ 21^2}$ = 34.64 $m^3/m^2$/day

Which is in the range of 30-60 $m^3/m^2$/day, Therefore O.K

Weir loading = $\frac{Flow}{Weir\ length} = \frac{12\ X\ 10^6}{1000\ X\ \pi\ X\ 21}$

= 182 $m^3$/m/day

Which is 100-250 $m^3$/m/day range, therefore O.K

Allowing 0.6m width for effluent channel, overall diameter of the tank = 21+0.6+0.6 = 22.22 m

Allowing a free board of 0.6m, total depth of tank = 3+0.6 = 3.6m

Provide a bottom slope of 10% from periphery to centre.

However provide two tanks of the above dimensions, one being as standby unit.

**Problem**

Design a continuous flow type rectangular primary sedimentation tank fitted with mechanical sludge cleaning equipment for treating the sewage from a city having a population of 80000 persons which has an assured water supply rate of 100 lpcd.

Assume the maximum flow to be 1.4 times the average flow. The necessary design parameters may be assumed. Sketch the designed sedimentation tank

**Answer:**

Given:

Population = 80000

Rate of water supply = 100 lpcd

Peak flow = 1.4 X average

Considering 80% of water supplied is coming out as sewage,, average sewage produced per day

$= \frac{80000 \ X \ 100 \ X \ 80}{1000 \ X \ 100} = 6400 \text{ m}^3$

Hourly flow = 6400/24 = 266.67 $m^3$

Assuming a detention period of 2 hours, the capacity of the tank required C = 266.67 X 2

C = 533.33 $m^3$

Assuming the liquid depth as 3m, the surface area of the tank = 533.33 / 3 = 177.78 $m^2$ say 178 $m^2$

Assuming the ratio of length to width as 2,

L X B = A or 2B X B = 178 $m^2$

Therefore Breadth = 178/2 = 9.43 m (say 9.5 m)

Length of tank = 2 X 9.5 = 19 m

Provide a bottom slope of 1% since for cleaning of sludge mechanical devices are employed towards the inlet end, where it is collected in a hopper trough of 0.6 m width.

Provide a free board of 0.5 m and therefore total depth = 3.5 m

Therefore the overall dimensions of the tank are

Length = 19 m, Width = 9.5 m

Overall depth = 3.5 m

Check:

i. Overflow rate:

$V_o$ = flow / surface area = $\frac{6400\frac{m^3}{day}}{19\ X\ 9.5\ m^2}$

= 35.45 $m^3/m^2/day$

Which is in the range of 30-45 $m^3$

Therefore design is safe.

ii. Weir loading:

= Flow/perimeter = $\frac{6400}{2(19+9.5)}$ = 112.28 $m^3/m/day$

Therefore O.K

**Problem**

Design a set of two circular sedimentation tanks to treat 5 million litre of sewage per day. Assume depth of 2.5 m check for hydraulic and weir loading.

**Solution**

Sewage flow = 5 X $10^6$ litres/day = 5000 $m^3/day$

Assuming a detention period of 2 hours, the total capacity of tank = $\frac{5000\ X\ 2}{24}$ = 416.67 $m^3$

Providing two circular tanks, capacity of each tank = 416.67/2 = 208.335 $m^3$

Assuming a depth 2.5 m, the surface area of each tank = 208.335/2.5 = 83.334 $m^2$

i.e., $\pi d^2/4$ = 83.334 $m^2$

Therefore diameter of each tank, d = $\frac{\sqrt{4\ X\ 83.334}}{\pi}$ = 10.3 m

Therefore provide two circular tanks of 10.5 m diameter and 2.5 m depth + 0.5 m free board and a bottom slope of 10% towards centre.

Check

Overflow rate = Flow/surface area = $\frac{5000}{\frac{2(\pi\ X\ 10.5^2)}{4}}$

$V_o = 28.87\ m^3/m^2/day$

Which is less than 30 $m^3/m^2/day$

Weir loading rate = Flow/weir length = $\frac{5000}{2(\pi X 10.5)}$

$= 75.79\ m^3/m/day$

Which is less than 100 $m^3/m$ of weir length/day

Therefore design is O.K

**Problem:**

Design a circular settling tank for primary treatment of domestic sewage for a flow of 15 MLD. Assume suitable values of hydraulic retention time and surface loading rate.

**Answer**

Capacity = 1250 $m^3$
Surface area = 416.67 $m^2$
Diameter = 23.0 m
Depth = 3m
Weir loading = 208 $m^3/m/d$

**Problem:**

Design a circular settling tank for primary treatment of domestic sewage for a flow of 12 MLD. Assume suitable values of hydraulic retention time and surface loading rate.

**Answer:**

Assumption of suitable hydraulic retention rate and surface loading rate
Diameter of the tank, d = 20.6 m
Capacity of tank, C = 1000 $m^3$
Depth of tank, D = 3 m
Check for weir loading

**Problem:**

Design a set of primary clarifiers for a town having a population of one lakh. The flow is 130 lpcd. Check the hydraulic loading rate and weir loading rate.

**Answer:**

Discharge, Q = 13 MLD
Flow through each tank = 6500 $m^3/d$
Assume, Detention time = 2 hrs
Capacity, C = 541.66 $m^3$
Assume, Depth = 3 m
Area, A = 180.55 $m^2$
Diameter of tank, d = 15.2 m
Hydraulic loading rate = 36 $m^3/m^2/d$
Weir loading rate = 136.19 $m^3/m/d$

# Unit 7

## Secondary Treatment

Wastewater leaving the primary settling tank has lost about 40 to 60 % of the suspended matter contained in the sewage, but it still has a high BOD. The BOD must be reduced further to ensure there is little adverse effect on the dissolved oxygen concentration in the receiving water course. Secondary treatment processes are designed specifically to reduce the oxygen demand of wastewaters. EPA defines a secondary treatment process as one that reduces the BOD and suspended solids levels to below 30 mg / L over a monthly average. Most secondary treatment processes use microbial action to reduce the oxygen demand of the waste. That is, the microorganisms within the process use oxygen - demanding materials as nutrients and energy for growth.

**Components of a Secondary Treatment Process**
Components of a secondary treatment process can be listed as follows;
(1) Microorganisms,
(2) $O_2$ supply (aeration),
(3) Wastewater and
(4) Mixing to bring all the other components together.

**Classification of Secondary Treatment Process**
Classification of secondary treatment processes is often based on the nature of microbial growth.
Organisms can be suspended in wastewater, or they can be attached to an inert surface.
If the micro-organisms are suspended in the WW during biological operation

Suspended growth processes

- Recycling of settled biomass is required.
- While the micro-organisms that are attached to a surface over which they grow

Attached growth processes

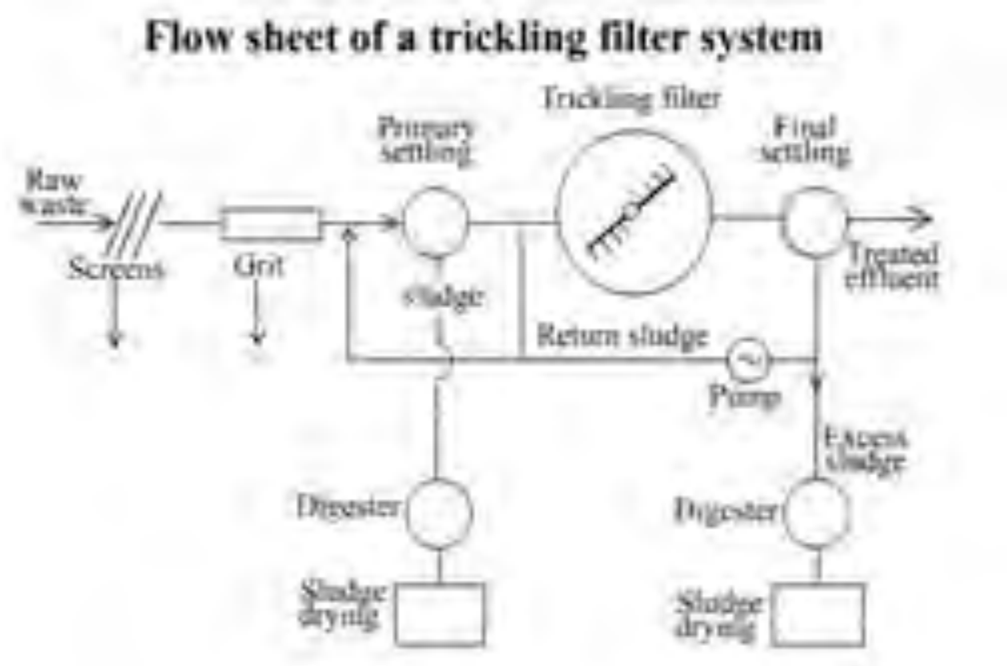
- The biomass attached to media (ex. rock, plastic, wood)
- Recycling of settled biomass is not required.

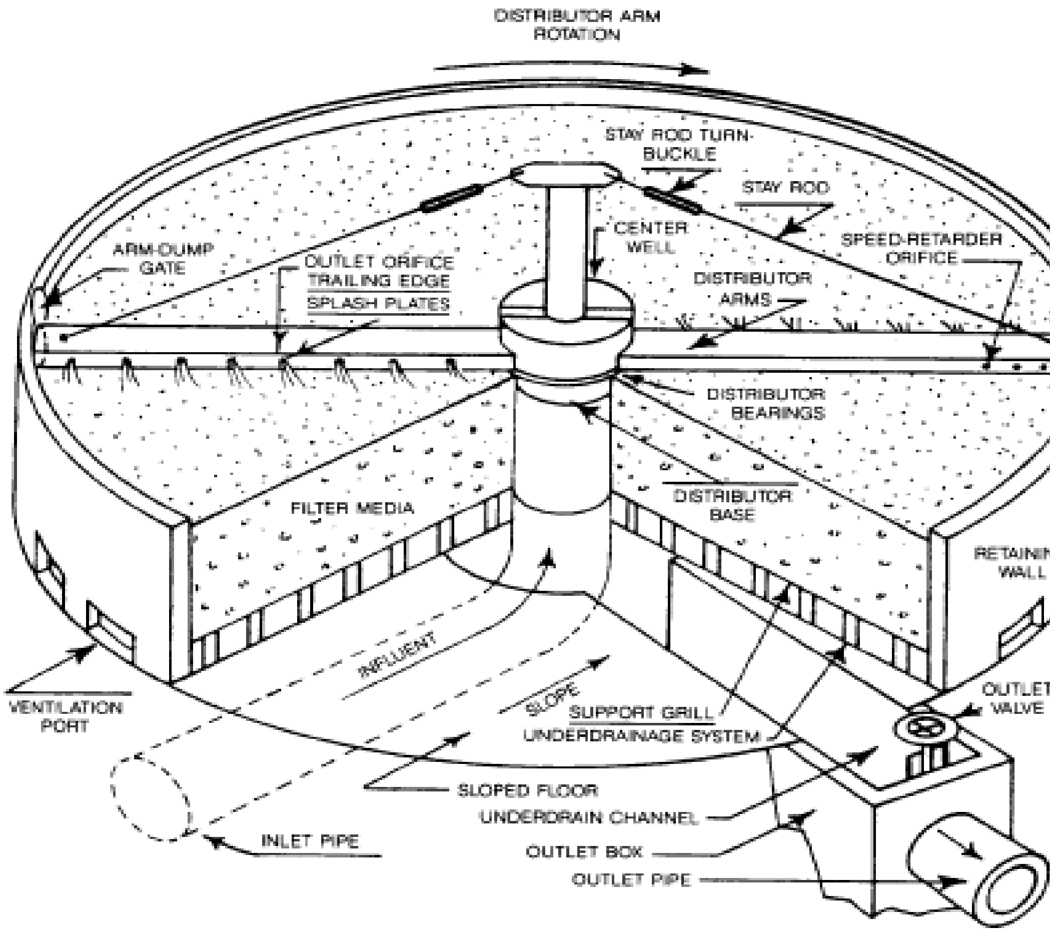
**Suspended Growth Processes:**
Activated Sludge (most common); (1) Conventional (tapered aeration), (2) Step aeration, (3) Contact stabilization, (4) Extended aeration, (5) High purity oxygen and (6) Oxidation ditch.

**Attached Growth Processes:** Attached growth processes can be listed as follows;
(1) Trickling filter (2) Rotating disks and (3) Biological towers.

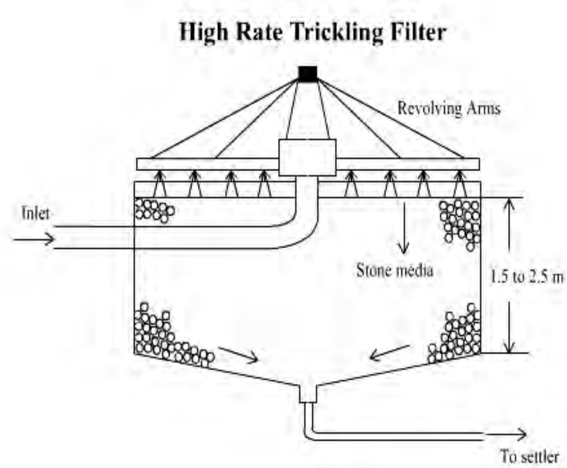
**Lagoons:** Lagoons can be listed as follows; (1) Mechanically aerated and (2) Waste stabilization ponds.

## Trickling filters






**Section of Trickling Filter**



Trickling filter is an ***attached growth process*** i.e. process in which microorganisms responsible for treatment are attached to an inert packing material. Packing material used in attached growth processes include rock, gravel, slag, sand, redwood, and a wide range of plastic and other synthetic materials.

A trickling filter consists of a bed of highly permeable media on whose surface a mixed population of microorganisms is developed as a *slime layer*. An attached growth process is happening in a trickling filter. Passage of wastewater through the filter causes the development of a gelatinous coating of bacteria, protozoa and other organisms on the media. With time, the thickness of the slime layer increases preventing oxygen from penetrating the full depth of the slime layer. In the absence of oxygen, anaerobic decomposition becomes active near the surface of the media.

The trickling filter consists of several major components including distribution system, media, under drains, effluent channel, secondary settling tank, and recirculation pumps and piping. Each of these components has one or more purposes.

The construction of a trickling filter consists of a cylindrical structure made of concrete or brick, an inside honey combed wall and the filter media which consists of gravel of size 25 mm to 75mm. the depth of the filtering media may vary from 2 to 3m.The under drainage consists of vitrified clay blocks with adequate openings. It is usually laid at a slope of 1 in 300, so as to maintain the velocity of flow of effluent as 0.9 m/sec. the influent to the trickling filter enters the tank trough a dosing tank kept at a height to transport the sewage by gravity. It enters the trickling filter through a central column inside the filter and reaches the rotating arms which has openings or spray nozzles through which the influent is sprayed or trickled to the bed. The rotating arm rotates at a speed of 2rpm. The media and the arms are separated by a distance of minimum, 15 to 20 cm to ensure good air circulation, it also get a good air flow through the honey combed wall and the under drainage system.

In operation, wastewater is distributed evenly over the surface of the trickling filter media. As the wastewater flows over the surface of the media the organisms in the slime remove the organic matter from the flow.

**Step by Step Process Description of Trickling Filter**

- The wastewater in trickling filter is distributed over the top area of a vessel containing non-submerged packing material.
- Air circulation in the void space, by either natural draft or blowers, provides oxygen for the microorganisms growing as an attached biofilm.
- During operation, the organic material present in the wastewater is metabolised by the biomass attached to the medium. The biological slime grows in thickness as the organic matter abstracted from the flowing wastewater is synthesized into new cellular material.
- The thickness of the aerobic layer is limited by the depth of penetration of oxygen into the microbial layer.
- The micro-organisms near the medium face enter the endogenous phase as the substrate is metabolised before it can reach the micro-organisms near the medium face as a result of increased thickness of the slime layer and loose their ability to cling to the media surface. The liquid then washes the slime off the medium and a new slime layer starts to grow. This phenomenon of losing the slime layer is called ***sloughing***.
- The sloughed off film and treated wastewater are collected by an under drainage which also allows circulation of air through filter. The collected liquid is passed to a settling tank used for solid- liquid separation.

**Filter Classification**

Trickling filters are classified by hydraulic or organic loading, as high-rate or low-rate.

| S.No. | Design Feature | Low Rate Filter | High Rate Filter |
|---|---|---|---|
| 1. | Hydraulic loading, $m^3/m^2.d$ | 1 - 4 | 10 - 40 |
| 2. | Organic loading,kg BOD / $m^3.d$ | 0.08 - 0.32 | 0.32 - 1.0 |
| 3. | Depth, m. | 1.8 - 3.0 | 0.9 - 2.5 |
| 4. | Recirculation ratio | 0 | 0.5 - 3.0 (domestic wastewater) upto 8 for strong industrial wastewater. |

Low-rate filters or conventional filters are relatively simple treatment units that normally produce a consistent effluent quality even with varying influent strength. Depending upon the dosing system, wastewater is applied intermittently with rest periods which generally do not exceed five minutes at the designed rate of waste flow. With proper loadings the low-rate trickling filter, including primary and secondary sedimentation units, should remove from 80 to 85 percent of the applied BOD.

High-rate filters are usually characterized by higher hydraulic and organic loadings than low-rate filters. The higher BOD loading is accomplished by applying a larger volume of waste per acre of surface area of the filter.

One method of increasing the efficiency of a trickling filter is to incorporate recirculation. Recirculation is a process by which the filter effluent is returned to and reapplied onto the filter. This recycling of the effluent increases the contact time of the waste with the microorganisms and also helps to "seed" the lower portion of the filter with active organisms.

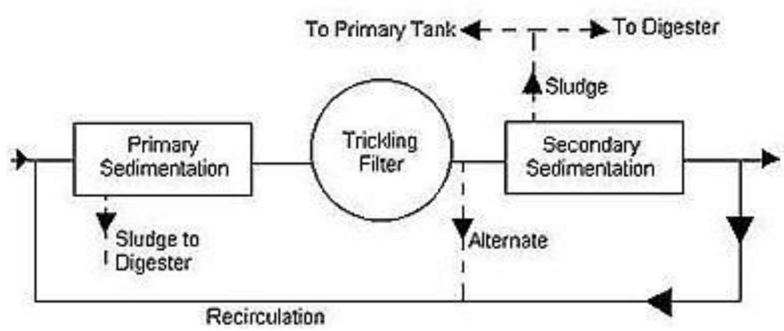
When recirculation is used, the hydraulic loading per unit area of filter media is increased. As a result, higher flow velocities will usually occur causing a more continuous and uniform sloughing of excess growths. Recirculation also helps to minimize problems with ponding and restriction of ventilation.

Recirculation can be continuous or intermittent. Return pumping rates can either be constant or variable. Sometimes recycling can be practiced during periods of low flow to keep the distributors in motion, to prevent the drying of the filter growths, and to prevent freezing during colder temperatures. Also, recirculation in proportion to flow may be utilized to reduce the organic strength of the incoming wastes, and to smooth out diurnal flow variations.

Recirculation can be accomplished by various techniques. Some of which are as follows:

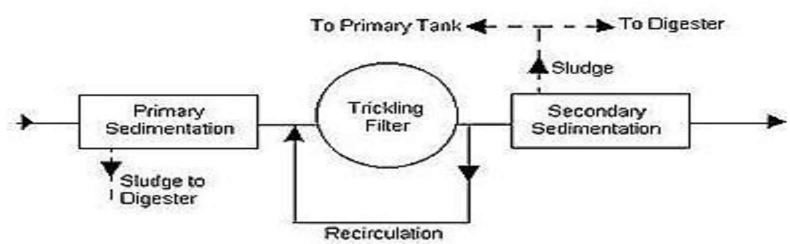
**Biofilter**: The bio-filter is a high-rate filter, usually 3 to 4 feet in depth, employing recirculation at all times. The recirculation in this case involves bringing the effluent of the filter or of the secondary sedimentation tank back through the primary settling tank. The secondary settling tank sludge is usually very light and can be continually fed back to the primary settling tank where the two types of sludges are collected together and pumped to the digester.

BIO-FILTER



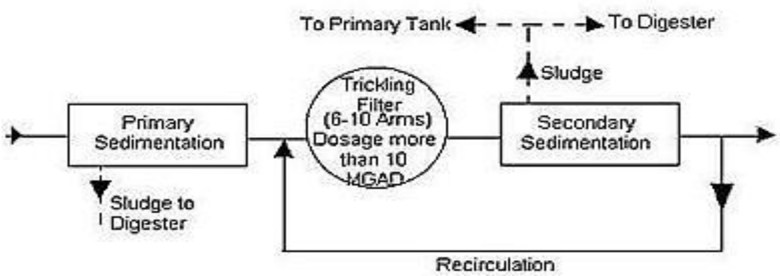

**Accelo-Filter**: The accelo-filter includes recirculation of unsettled effluent from the filter back to the inlet of the filter distributor. It is used for both low-rate and high-rate filters, the former being applicable if a well nitrified effluent is required.

ACCELO-FILTER




**Aero Filter**: The aero-filter is still another process which distributes the wastewater by maintaining a continuous rain-like application of the wastewater over the filter bed. For small beds, distribution is accomplished by a disc distributor revolving at a high speed of 260 to 369 rpm set 20" above the surface of the filter to give a continuous rain-like distribution over the entire bed. For large beds a large number of revolving distributor arms, 10 or more, tend to give more uniform distribution. These filters are always operated at a rate in excess of 10 million gallons per acre of surface area per day.

AERO-FILTER




**Common Problems**

Due to its simple design, in actual operation the trickling filter is one of the most trouble-free types of secondary treatment processes. It requires much less operating attention and process control than the activated sludge system, but some problems do exist. The following is a summary of some of the more common problems and cures:

a) **Ponding** can cause odors and decrease filter efficiency. Ponding is normally the result of: (a) excessive organic loading without a corresponding higher recirculation rate,
(b) Use of media which is too small,
(c) Clogging of under drain system,
(d) Non-uniform media size or breaking up of media, and
(e) Trash or debris in filter voids.

**Minor Ponding** can be eliminated by:

1. Spraying the surface with high pressure water hose.
2. Stirring or agitating ponding area with stick, rake, etc.
3. Dousing the filter with chlorine. Applying chlorine to a ponding filter by chlorinating at the dosing tank to produce a residual of about one to two mg/L at the nozzles may help reduce ponding. Chlorination is continued until the filters are free. There may be some deterioration of efficiency of the filters during chlorination. Obviously, if ponding is caused by the size of the media, chlorination will be of no benefit. If the ponding is caused by overloading, chlorination may be of temporary benefit. If ponding was caused by excessive

growths, this deteriorating condition will usually not return until conditions, such as temperature, that caused the excessive growth are repeated.

4. Flooding filter and keeping the media submerged for approximately 24 hours will sometimes cause the growth to slough. Growths become anaerobic and tend to release from media.
5. Shutting off the flow to the filter. The growths will die and tend to be flushed out when the unit is put back into service.

**b) Odors**. Since the trickling filter is an aerobic process, no serious odors should exist. If foul odors are present, anaerobic conditions are the most likely cause. Anaerobic conditions usually predominate next to the media surface. If the surface of the slime growth is aerobic, odors should be minimal. If odors are present, corrective action should be taken immediately or the condition could get worse. Some corrective measures are:

1. Try to maintain aerobic condition in the collection system and in the primary treatment units.
2. Check the ventilation of the filter for clogging and stoppages.
3. Check the under drain system for clogging and stoppages.
4. Increase recirculation rate; this usually provides added oxygen to the filter and may increase sloughing.
5. Keep wastewater in filter; do not allow it to splash on exposed surfaces, weeds, or grass.
6. Add odor-masking agents.
7. Pre-chlorination at primary tank influent or at the dosing tank. The dose used is not sufficient to produce residual chlorine but only to destroy the odors. Chlorination to a residual of less than 0.5 mg/L normally does not interfere with the activity of the living organisms and thus does not affect the purification obtained by the operation of a trickling filter. However, chlorination of a trickling filter influent cannot be used until after the filter has been in active operation. Except in a large plant, the chlorine dose is generally set at about half the chlorine demand and not adjusted for moderate variations in flow or strength.

**c) Filter Flies** are a nuisance to plant personnel and nearby neighbors. These tiny, gnat-size flies are called psychoda. They are occasionally found in great numbers, preferring an alternate wet and dry environment for development. The flies are most frequently found in low or standard rate filters with an intermittent dosing system.

Control can be accomplished by:

1. Increasing recirculation. A continuous waste flow to the filter will tend to wash fly larvae from the filter.
2. Flushing the side walls of the filter by opening the flap valve at the end of the distributor arm.
3. Flooding the filter intermittently to prevent completion of the fly life cycle. This life cycle can be as short as seven days in warm weather. Filters should be flooded for approximately 24 hours.
4. The addition of chlorine, which is toxic to the flies and larvae.
5. Keeping the plant grounds neat, clean and free from excessive weeds, plants, and grass, which are excellent breeding grounds for the flies.

d) **Weather Problems**. Cold weather can cause an occasional build-up of ice on the media, walls, distributor arms and orifices, resulting in operating problems and loss of efficiency. During cold temperatures, the organisms metabolic process slow down and as a result efficiency decreases.

Measures which can be implemented to reduce cold weather problems are:

1. Decrease the recirculation rate to prevent splashing at distributor arm, but maintain sufficient flow to keep the filter working.
2. Adjust orifices at splash plates to reduce the spraying effect.
3. Construct wind screens or covers to reduce heat loss.
4. Break up any ice build-up.
5. Partially open flap gates at end of distributor arm to allow for a stream of water rather than a spray of water.

**High Rate Trickling Filter**

High-rate trickling filters, including primary and secondary sedimentation, should, under normal operation, remove from 65 to 85 percent of the BOD of the wastewater. Recirculation should be adequate to provide continuous dosage at a rate equal to or in excess of 10 million gallons per acre per day. As a result of continuous dosing at such high rates, some of the solids accumulated on the filter medium are washed off and carried away with the effluent continuously.

Conventional filters are modified to make high rate filters using the following modifications:

1. Better quality filtering media is used, like larger size stone media or plastic media.
2. Depth of filter media is limited to 1.5 to 2m
3. Size of under drain is increased and their slope is also made steeper so that the filter effluent can be collected and conveyed quickly.
4. Speed of rotating arm is increased.
5. Size of secondary settling tank is also increased.

High-rate trickling filters have been used advantageously for pretreatment of industrial wastes and unusually strong wastewaters. When so used they are called "roughing filters".

Generally, most organic wastes can be successfully treated by trickling filtration. Normally food processing, textile, fermentation and some pharmaceutical process wastes are amenable to trickling filtration.

**Pretreatment**

The beginning of the trickling filter treatment process is pretreatment. The wastewater passes through a bar screen to remove large objects. Then the influent is passed through a grit screen or chamber to remove inorganic materials such as eggshells, corn, sand, and tissue. The comminutor shreds the solid material into smaller pieces, which allows the solids to enter the plant without causing mechanical problems or clogging the pumps.

**Separator and Digester**

In many treatment systems, two clarifiers are used. The **primary clarifier** follows the grit chamber while the secondary clarifier follows the aerator, oxidation ditch, trickling filter, or other type of biological treatment.

In the case of a trickling filter system, the water from the grit chamber enters a **separator**, which acts as a primary clarifier. Here, organic solids are separated out of the water using changes in velocity. The mostly liquid portion of the organic matter goes on to the trickling filter while the mostly solid portion goes to the anaerobic digester.

**Recirculation of Treated Sewage**

One method of increasing the efficiency of a trickling filter is to incorporate recirculation. *Recirculation is a process by which the filter effluent is returned to and reapplied onto the filter.* This recycling of the effluent increases the contact time of the waste with the microorganisms and also helps to "seed" the lower portion of the filter with active organisms.

When recirculation is used, the hydraulic loading per unit area of filter media is increased. As a result, higher flow velocities will usually occur causing a more continuous and uniform sloughing of excess growths. Recirculation also helps to minimize problems with ponding and restriction of ventilation.

Recirculation can be continuous or intermittent. Return pumping rates can either be constant or variable. Sometimes recycling can be practiced during periods of low flow to keep the distributors in motion, to prevent the drying of the filter growths, and to prevent freezing during colder temperatures. Also, recirculation in proportion to flow may be utilized to reduce the organic strength of the incoming wastes, and to smooth out diurnal flow variations.

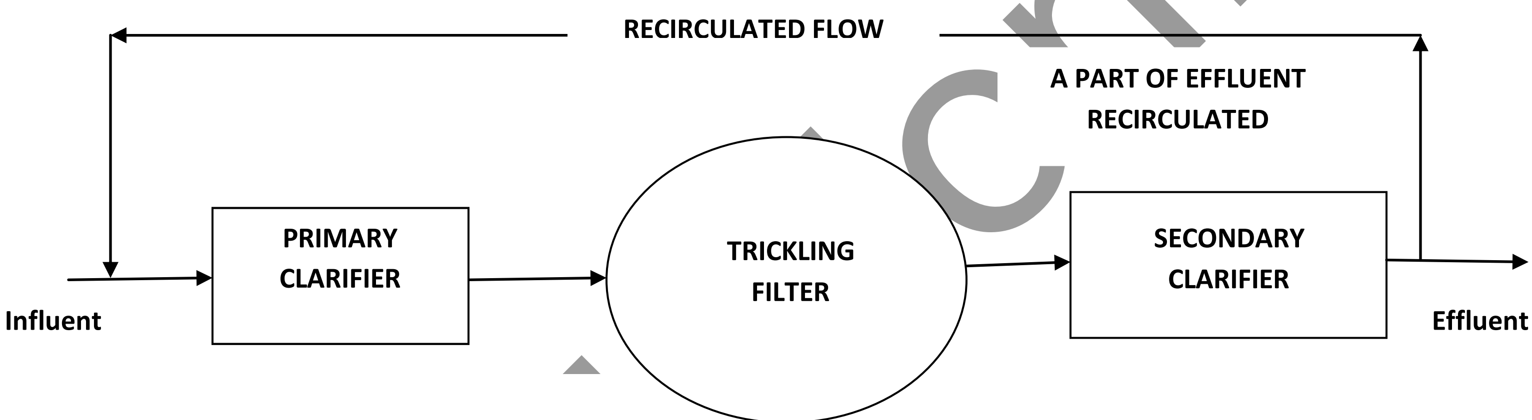
Recirculation is advantageous in following ways

a. It keeps self propelled distributors running at the time of reduced flows
b. Thickness of biological film on contact media is reduced by forced film sloughing.
c. Filter influent is freshened due to which foul odor is prevented.

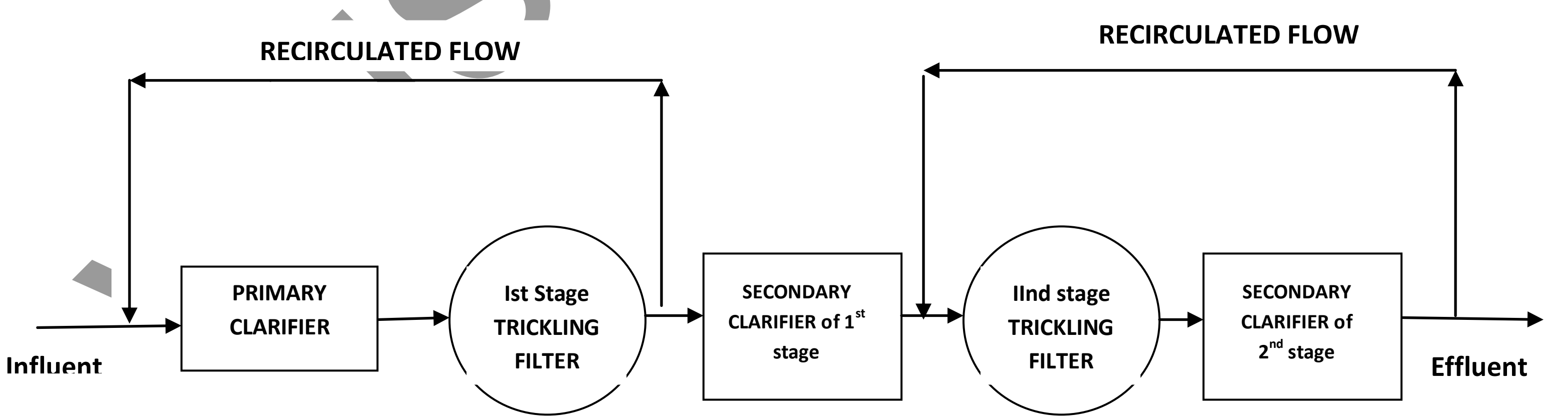
d. Applied sewage is seeded with active organisms and enzymes of effluent, due to which the efficiency of filter is increased.

**Single stage filter and Two stage filter**

High rate filters may be single stage or two stage. Single stage filters consists of primary settling tank, the filter, secondary settling tank and facilities for recirculation of the effluent. Recirculation is from secondary settling tank to the primary settling tank or effluent from filter itself before it enters the secondary clarifier may be sent back to the primary clarifier.




Two stage filters consists of two filters in series with a primary settling tank, an intermediate settling tank, and a final settling tank. Effluent from first stage filter is applied on the second stage filter.



Generally trickling filter design is based on empirical relationships to find the required filter volume for a designed degree of wastewater treatment. Types of equations:

1. NRC equations (National Research Council of USA)
2. Rankins equation
3. Eckenfilder equation
4. Galler and Gotaas equation

NRC and Rankin's equations are commonly used. NRC equations give satisfactory values when there is no re-circulation, the seasonal variations in temperature are not large and fluctuations with high organic loading. Rankin's equation is used for high rate filters.

***NRC equations*:** These equations are applicable to both low rate and high rate filters. The efficiency of single stage or first stage of two stage filters, $E_2$ is given by

$$E_2 = \frac{100}{1+0.44(F_{1.BOD}/V_1.Rf_1)^{1/2}}$$

For the second stage filter, the efficiency $E_3$ is given by

$$E_3 = \frac{100}{[(1+0.44)/(1-E_2)](F_{2.BOD}/V_2.Rf_2)^{1/2}}$$

where $E_2$= % efficiency in BOD removal of single stage or first stage of two-stage filter, $E_3$=% efficiency of second stage filter, $F_{1.BOD}$= BOD loading of settled raw sewage in single stage of the two-stage filter in kg/d, $F_{2.BOD}= F_{1.BOD}(1- E_2)$= BOD loading on second-stage filter in kg/d, $V_1$= volume of first stage filter, $m^3$; $V_2$= volume of second stage filter, $m^3$; $Rf_1$= Recirculation factor for first stage, $R_1$= Recirculation ratio for first stage filter, $Rf_2$= Recirculation factor for second stage, $R_2$= Recirculation ratio for second stage filter.

**Rankins equation:** This equation also known as Tentative Method of Ten States USA has been successfully used over wide range of temperature. It requires following conditions to be observed for single stage filters:

1. Raw settled domestic sewage BOD applied to filters should not exceed 1.2 kg $BOD_5$/day/ $m^3$ filter volume.
2. Hydraulic load (including recirculation) should not exceed 30 $m^3/m^2$ filter surface-day.
3. Recirculation ratio (R/Q) should be such that BOD entering filter (including recirculation) is not more than three times the BOD expected in effluent. This implies that as long as the above conditions are satisfied efficiency is only a function of recirculation and is given by:

$$E = \frac{(R/Q) + 1}{(R/Q) + 1.5}$$

## Trickling Filter Design

**Problem**:

Design a low rate filter to treat 6.0 Mld of sewage of BOD of 210 mg/l. The final effluent should be 30 mg/l and organic loading rate is 320 $g/m^3/d$.

**Answer:**

Assume 30% of BOD load removed in primary sedimentation i.e., = 210 x 0.30 = 63 mg/l.
Remaining BOD = 210 - 63 = 147 mg/l.
Percent of BOD removal required = (147-30) x 100/147 = 80%
BOD load applied to the filter = flow x conc. of sewage (kg/d) = $6 \times 10^6 \times 147/10^6$ = 882 kg/d
To find out filter volume, using NRC equation

$$E_2 = \frac{100}{1+0.44(F_{1.BOD}/V_1.Rf_1)^{1/2}}$$

$$80 = \frac{100}{1+0.44(882/V_1)^{1/2}} \quad Rf_1 = 1, \text{ because no circulation.}$$

$V_1 = 2704\ m^3$
Depth of filter = 1.5 m, Fiter area = 2704/1.5 = 1802.66 $m^2$, and Diameter = 48 m < 60 m
Hydraulic loading rate = $6 \times 10^6/10^3 \times 1/1802.66 = 3.33 m^3/d/m^2 < 4$ hence o.k.
Organic loading rate = 882 x 1000 / 2704 = 326.18 $g/d/m^3$ which is approx. equal to 320.

**Problem:**

Determine the depth, volume and efficiency of a standard rate trickling filter for the following data:

1. Quantity of settled sewage = 8 million litres/day.
2. BOD of sewage = 225 mg/l.
3. Rate of organic loading = 225 $gm/m^3/day$.
4. Rate of surface loading = 2000 $l/m^2/day$.

**Answer:**

Volume of standard rate trickling filter = 8000 $m^3$
Area of trickling filter = 4000 $m^2$
Depth = 2 m
Efficiency = 82.73 %

**Problem:**

Determine the depth and volume of a standard rate trickling filter for the following data:

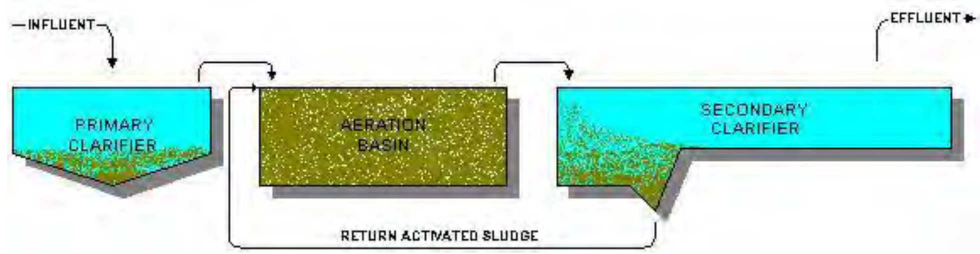
i. Quantity of settled sewage = 6 MLD
ii. BOD of sewage = 200 mg/l
iii. Rate of organic loading = 200 $gm/m^3/day$

Rate of surface loading = 2500 $l/m^2/day$

**Answer:**

Volume of a standard rate trickling filter, $V = 6X10^3\ m^3$
Area, $A = 2.4X10^3\ m^2$
Depth, D = 2.5 m

**Activated Sludge Process**

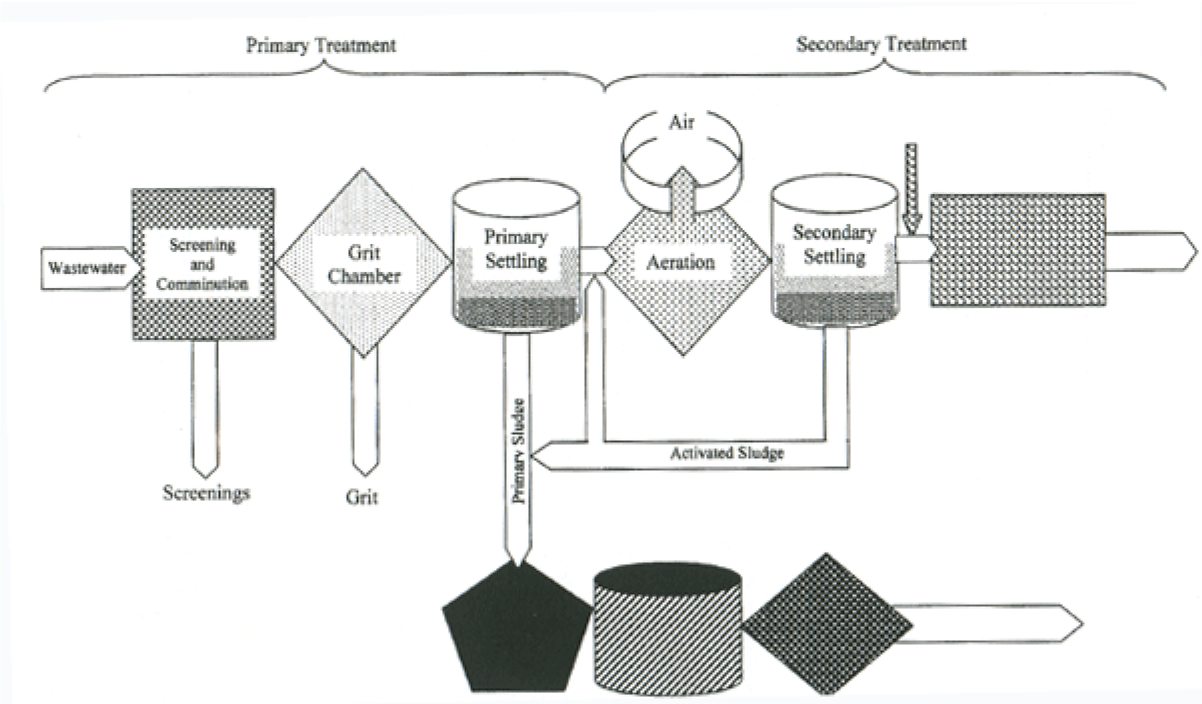



Activated sludge refers to biological treatment processes that use a suspended growth of organisms to remove BOD and suspended solids. As shown below, the process requires an aeration tank and a settling tank.

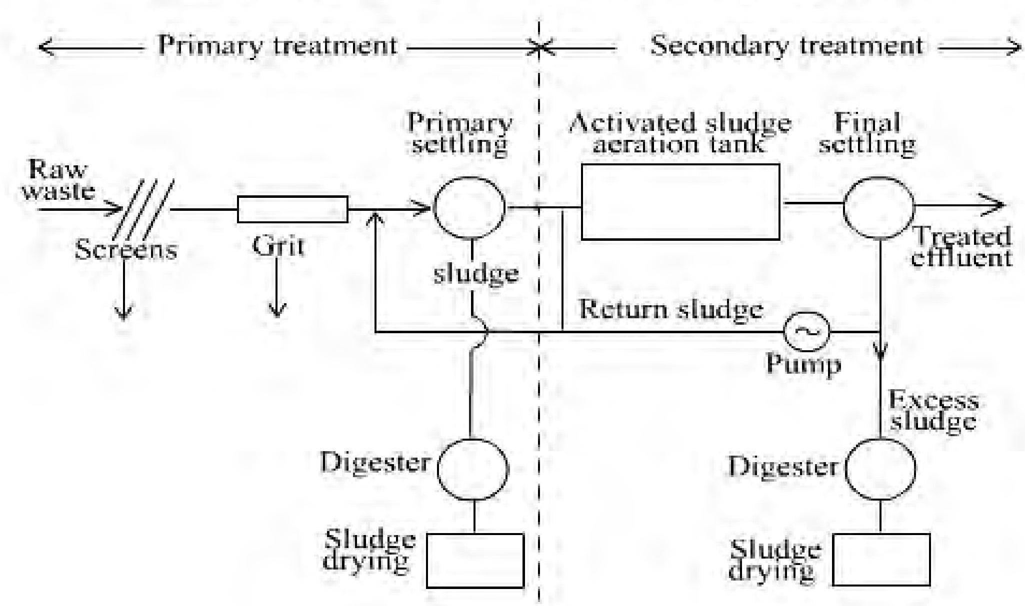
The activated sludge process was developed in England in 1914 and was so named because it involved the production of an activated mass of microorganisms capable of aerobically stabilizing the organic content of a waste. Activated sludge is probably the most versatile of the biological treatment processes capable of producing an effluent with any desired BOD. The process has thus found wide application among domestic wastewater and industrial wastewater treatment.

Like the trickling filter, activated sludge is a biological contact process where bacteria, fungi, protozoa and small organisms such as rotifers and nematode worms are commonly found. The bacteria are the most important group of microorganisms for they are the ones responsible for the structural and functional activity of the activated sludge floc. All types of bacteria make up activated sludge. The predominate type of bacteria present will be determined by the nature of the organic substances in the wastewater, the mode of operation of the plant, and the environmental conditions present for the organisms in the process.

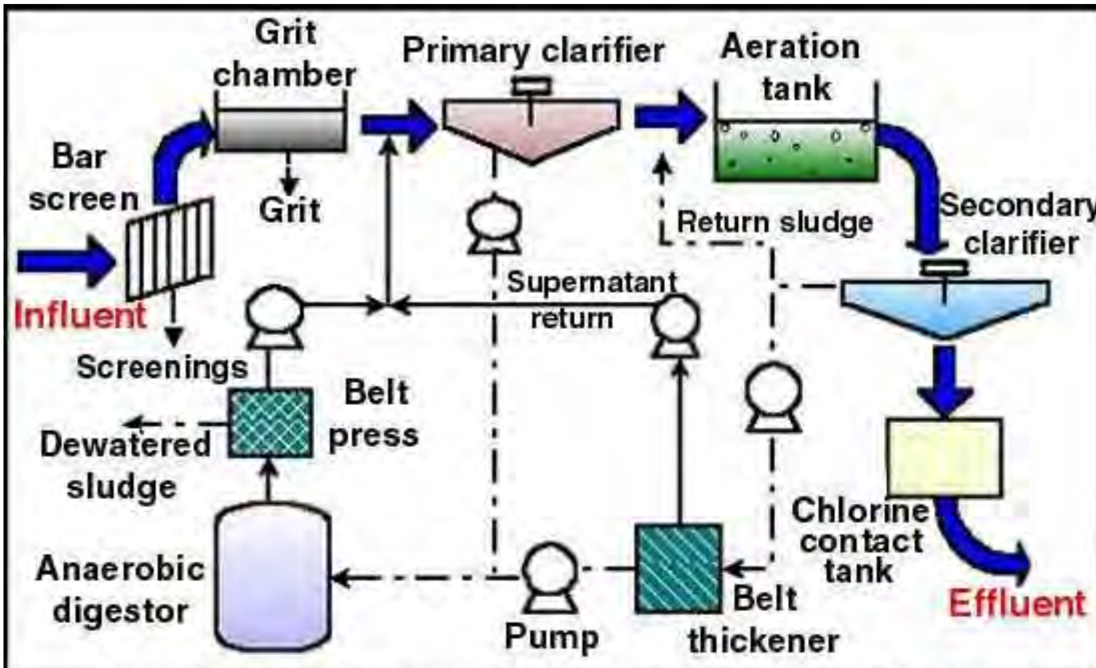
Fungi are relatively rare in activated sludge. When present, most of the fungi tend to be of the filamentous forms which prevent good floc formation and therefore decrease the efficiency of the plant. A high carbohydrate waste, unusual organic compounds, low pH, low dissolved oxygen concentrations, and nutrient deficiencies stimulate fungi growths. The other forms of microorganisms present in activated sludge play a minor role in the actual stabilization of the organics in wastewater.




**Flow sheet of an activated sludge system**



In addition, support equipment, including return pumps, waste pumps, flow measurement devices for return and waste, as well as equipment to provide aeration (mixers and/or blowers) is also required.



Typical Activated Sludge Treatment Process Flow Diagram

Primary effluent is mixed with return activated sludge to form mixed liquor. The mixed liquor is aerated for a specified length of time. During the aeration the activated sludge organisms use the available organic matter as food producing stable solids and more organisms. The suspended solids produced by the process and the additional organisms become part of the activated sludge. The solids are then separated from the wastewater in the settling tank. The solids are returned to the influent of the aeration tank (return activated sludge). Periodically the excess solids and organisms are removed from the system (waste activated sludge). Failure to remove waste solids will result in poor performance and loss of solids out of the system over the settling tank effluent weir.

**Factors affecting ASP**

There are a number of factors that affect the performance of an activated sludge treatment system. These include:

- temperature

- return rates
- amount of oxygen available
- amount of organic matter available
- pH
- waste rates
- aeration time
- wastewater toxicity

To obtain desired level of performance in an activated sludge system, a proper balance must be maintained between the amount of food (organic matter), organisms (activated sludge) and oxygen (dissolved oxygen).

**Activated Sludge Process-Principle**

**Process Design Consideration**

The activated sludge process is usually employed following primary sedimentation. The wastewater contains some suspended and colloidal solids and when agitated in the presence of air, the suspended solids form nuclei on which biological life develop and gradually build up to larger solids which are known as activated sludge.

Activated sludge is a brownish floc-like substance consisting of organic matter obtained from the wastewater and inhabited by myriads of bacteria and other forms of biological life. Activated sludge with its living organisms has the property of absorbing or adsorbing colloidal and dissolved organic matter. The biological organisms utilize the absorbed material as food and convert it to inert insoluble solids and new bacterial cells. Much of this conversion is a step-by-step process. Some bacteria attack the original complex substances to produce simpler compounds as their waste products. Other bacteria use the waste products to produce sill simpler compounds and the process continues until the final waste products can no longer be used as food for bacteria.

The generation of activated sludge or floc in wastewater is a slow process and the amount so formed from any volume of wastewater during its period of treatment is small and inadequate for the rapid and effective treatment of the wastewater which requires large concentrations of activated sludge. Such concentrations are built up by collecting the sludge produced from each volume of wastewater treated and re-using it in the treatment of subsequent wastewater flows. The sludge so re-used is known as returned sludge. This is a cumulative process so that eventually more sludge has been produced and is available to maintain a viable biological population of organisms to treat the incoming wastes. The surplus, or excess activated sludge, is then permanently removed from the treatment process and conditioned for ultimate disposal.

The activated sludge must be kept in suspension during its period of contact with the wastewater being treated by some method of agitation. The activated sludge process, therefore, consists of the following steps:

1. Mixing the activated sludge with the wastewater to be treated (mixed liquor).
2. Aeration and agitation of this mixed liquor for the required length of time.
3. Separation of the activated sludge from the mixed liquor, in the final clarification process.
4. Return the proper amount of activated sludge for mixture with the wastewater.
5. Disposal of the excess activated sludge.

**Mixing the Activated Sludge with the Wastewater to be Treated**

The first step in the activated sludge process is to bring the microorganisms in contact with the organics of the wastewater. This is generally accomplished by the rapid mixing of the return sludge with the wastewater at the inlet of the aeration tank. In some cases small mixing chambers are provided, but this is not the common practice.

**Aeration and Agitation of Mixed Liquor**

Aeration serves at least three important functions: (1) mixing the returned sludge with effluent from primary treatment, (2) keeping the activated sludge in suspension, and (3) supplying the oxygen to the biochemical reactions necessary for the stabilization of the wastewater. The theoretical oxygen requirement can be computed by knowing the BOD of the waste, the amount of organisms wasted from the system per day and the degree of treatment (whether a nitrified effluent is required). For practical purposes, enough air should be added to the waste to maintain at least two mg/L of dissolved oxygen under all conditions of loading in all parts of the aeration tank. The air requirements for good treatment can be satisfied either by a diffused air system or by mechanical aerators.

In the diffused air system, air under low pressure, generally not more than eight to ten pounds, is supplied by blowers and forced through various types of porous material in plates or tubes installed near the bottom of the aeration tank. As the air is discharged into the liquid phase it breaks up into fine bubbles thus increasing the surface area across which oxygen diffuses from air into the wastewater. The plates or tubes are so located in the aeration tank that a rotary motion is imparted to the wastewater mixture resulting in a considerable amount of air being absorbed from the atmosphere. Diffuser plates may be composed of fused crystalline alumina or high-silica sand. They are set in containers usually made of reinforced concrete. Diffuser tubes are made of similar material, or, more recently, of corrugated stainless steel pipe with multiple outlets and wrapped with saran twisted cord. These are suspended in the aeration tank in sections and can be disconnected above the wastewater surface and removed for cleaning or renewal. When installed on swing joint connections so that they may be brought to the surface of the tank, they are known as "Swing Diffusers".

To prevent clogging of the diffuser plates or tubes, the air supplied to them should be filtered to remove dust, oil or other impurities, and the piping should be of non-corrosive material. There are a number of types of filters available based on different principles which may be used alone or in combination.

Mechanical aerators are of two general types -- surface and turbine. Surface aerators consists of submerged or partially submerged impellers, which are centrally mounted in the aeration tank and which agitate the wastewater vigorously, entraining air in the wastewater and causing a rapid change of the air-water interface to facilitate solution of the air. Another type of surface aerator, more popular in Europe, consists of a paddle wheel or brush, partly submerged in the wastewater, revolving on a horizontal axis. Air is absorbed by surface contact and by droplets thrown through the air by the paddle mechanism.

Turbine aerators are usually upflow types that rely on violent agitation of the surface and air entrainment for their efficiency. A draft tube may be utilized to control the flow pattern of the circulating liquid within the aeration tank. The draft tube is a cylinder with flared ends mounted concentrically with the impeller, and extending from just above the floor of the aeration tank to just beneath the impeller.

In the activated sludge process, the sludge accomplishes the major part of the removal of BOD from the wastewater being treated in a relatively short period of aeration. It takes, however, a much longer time for the sludge to assimilate the organic matter which it has absorbed. During this time an aerobic environment must be maintained. To effect the most complete treatment of wastewater and the most economical operation in the conventional activated sludge process, an aeration detention time of six to eight hours has been found to be adequate for diffused air aeration and nine to twelve hours for mechanical aeration. Substantially shorter periods are used in some of the modifications of the conventional process. These shorter aeration periods generally result in a lowering of the quality of the plant effluents.

**Separation of Activated Sludge from the Mixed Liquor**

The function of the secondary clarifier is to separate the activated sludge solids from the mixed liquor. These solids represent the colloidal and dissolved solids that were originally present in the wastewater. In the aeration unit they were incorporated into the activated sludge floc, which are settleable solids. The separation of these solids, a critical step in the activated sludge process, is accomplished in the secondary or final settling tanks. These tanks are similar in design to the mechanically cleaned primary sedimentation tanks but with a surface settling rate not to exceed 800 gallons per square foot per day.

The cycle of sludge removal from the secondary tanks is much more important than with primary tanks. Some sludge is being removed continuously to be used as returned sludge in the aeration tanks. The excess sludge must be removed before it loses its activity because of the death of the aerobic organisms resulting from lack of oxygen at the bottom of the tank. Anaerobic sludges in the final clarifier can cause "rising sludge". This should not be confused with a bulking sludge. Rising sludges are a result of denitrification and septicity. It is possible, where facilities are available, to reactivate return sludge in separate reaeration tanks before addition to the wastewater. However, it is much wiser to retain the activity of the sludge by prompt withdrawal from the tank.

**Return Sludge Requirements**

The purpose of return sludge is to maintain a concentration of activated sludge in the aeration tank sufficient for the desired degree of treatment. Ample return sludge pump capacity should be provided since the return sludge volume may range from 10 to 50 percent of the volume of wastewater being treated and sometimes more. For a conventional plant, the percentage is usually between 20 and 30 percent. The best concentration must be determined for each plant by trial operation and should be carefully maintained by controlling the proportion of return sludge. The maximum concentrations are limited by the air supply and wastewater load. If solids area allowed building up, the air and food requirements will exceed those available and an upset will occur.

**Sludge Wasting from Activated Sludge**

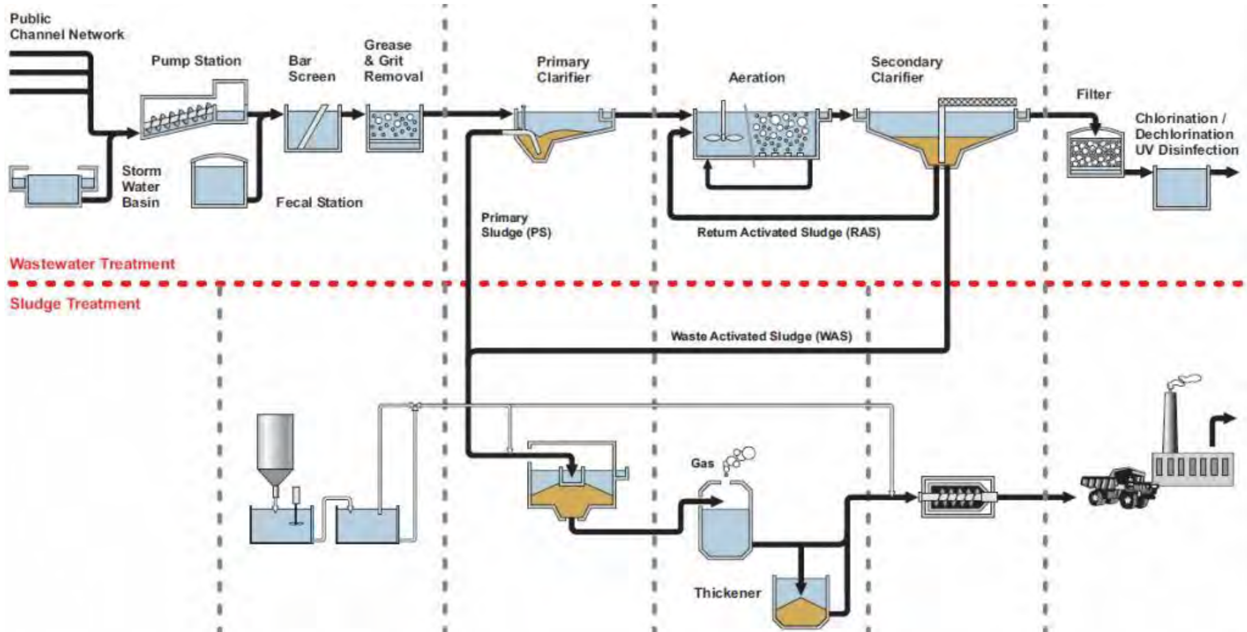
Excess activated sludge should be wasted as required to maintain the desired solids concentration in the aeration tank. This can be done by either withdrawing mixed liquor directly from the aeration tank or to waste from the sludge return line. The wasted mixed liquor can then be discharged to a thickening tank or to the primary tanks where the sludge settles and mixes with the raw primary sludge. The waste sludge, usually from the sludge return line, is further thickened by final sedimentation, centrifuging, or flotation thickening and then treated by biological or chemical means.

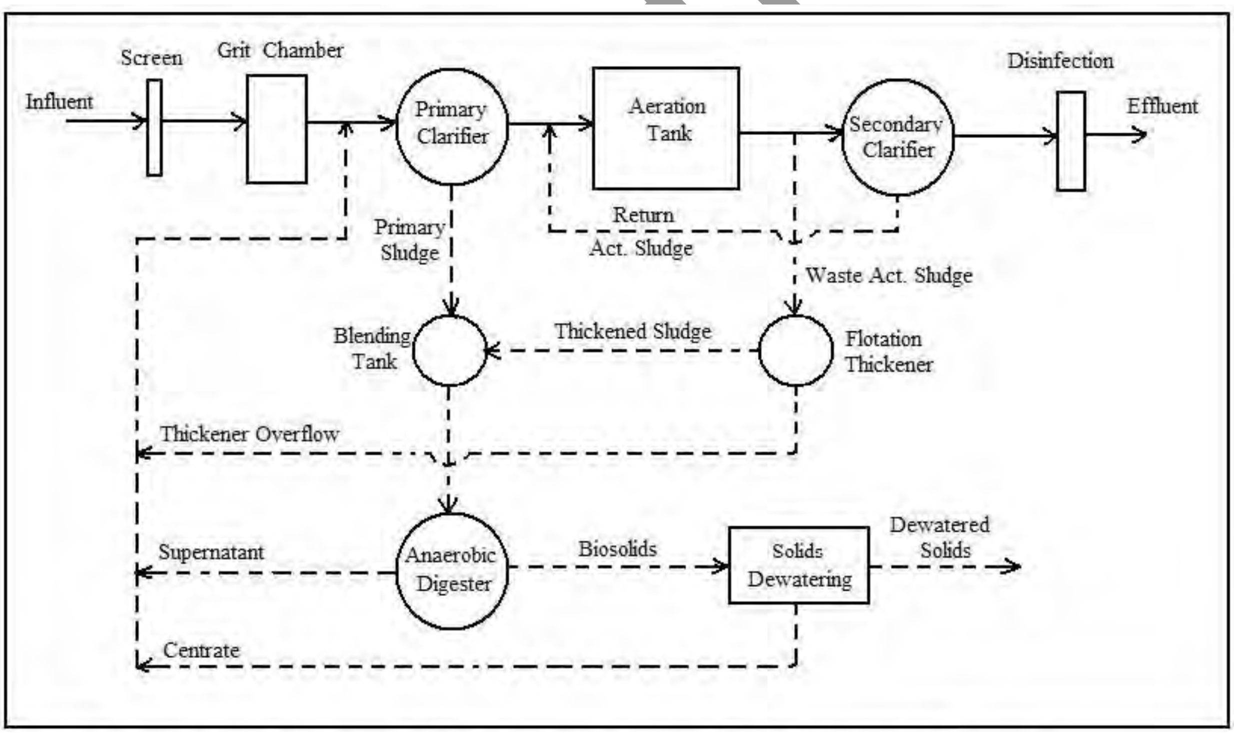
The above steps are essential in the operation of an activated sludge process. Nevertheless, the operator of a plant should have a large degree of freedom or flexibility to modify the operation of an activated sludge plant to achieve the required treatment. The modifications are expressed in terms of the level of solids in the aeration tank, concentration of the solids, method of wastewater introduction into the aeration tank, and in terms of aeration time.

**Typical Process Loading Ranges**

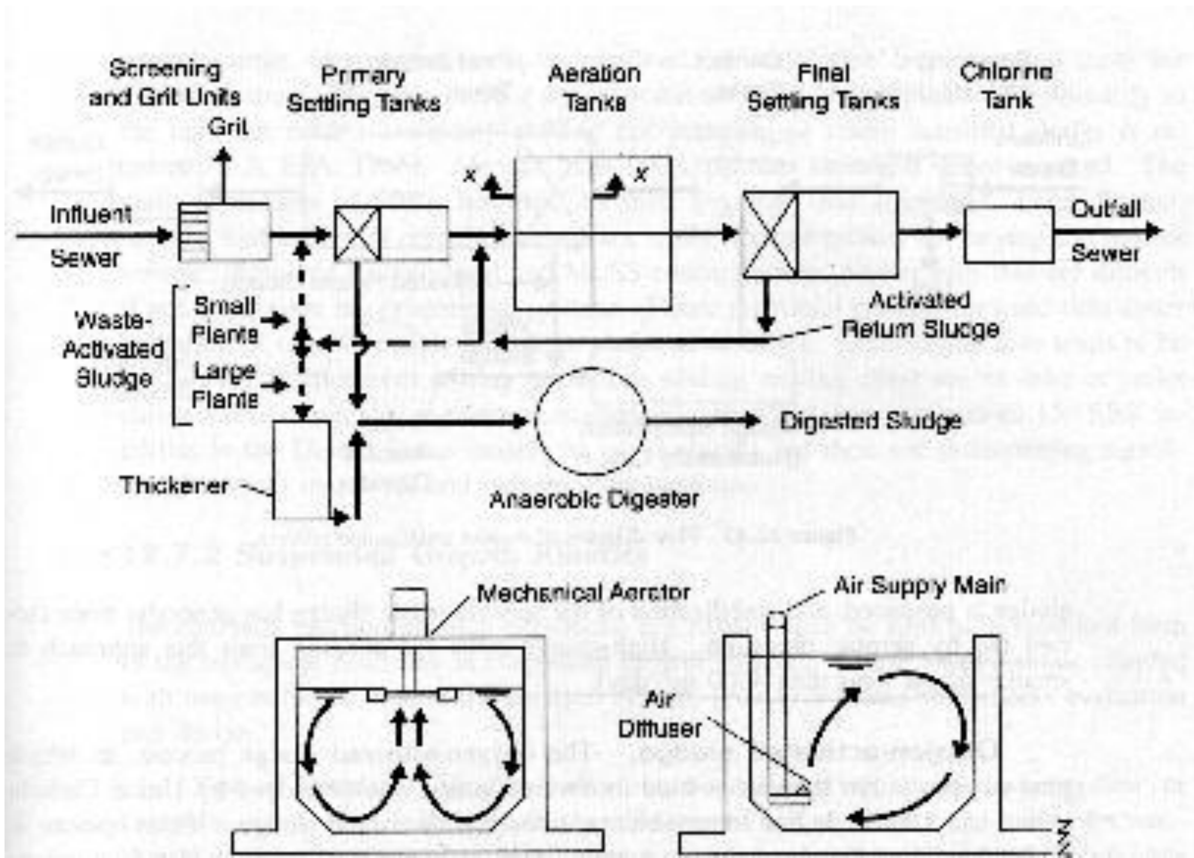
| Loading Range | SRT (day) | F : M (kg $BOD_5$/kg MLVSS.day) |
|---|---|---|
| High | 3 - 5 | 0.4 - 1.5 |
| Medium | 5 - 15 | 0.2 - 0.4 |
| Low | 15 - 30 | 0.05 - 0.2 |

## ASP Flow Diagram




Flow Diagram for Overall Solids Balance - Activated Sludge Wastewater Treatment




## Modifications of ASP

### Activated Sludge Modifications

Many activated sludge process modifications exist. Each modification is designed to address specific conditions or problems. Such modifications are characterized by differences in mixing and flow patterns in the aeration basin, and in the manner in which the microorganisms are mixed with the incoming wastewater.

The major process modifications of the activated sludge process are:

1. conventional
2. tapered aeration
3. complete mix
4. step aeration
5. contact stabilization

6. extended aeration
7. pure oxygen systems

**Conventional Modification**

This configuration requires primary treatment, has the influent and returned sludge enter the tank at the head end of the basin, mixing is accomplished by the aeration system, and provides excellent treatment. On the downside, this modification requires large aeration tank capacity, higher construction costs, high initial oxygen demand, and is very sensitive to operation problems, such as bulking.

**Tapered Aeration**

The tapered aeration system is similar to the conventional activated sludge process. The major difference is in the arrangement of the diffusers. The diffusers are close together at the influent end where more oxygen is needed. Toward the other end of the aeration basin, the spacing of the diffusers is increased.

**Step Aeration**

In step aeration, the returned sludge is applied at several points in the aeration basin. Generally, the tank is subdivided into three or more parallel channels with around-the-end baffles, and the sludge is applied at separate channels or steps. The oxygen demand is uniformly distributed.

**Complete Mix Aeration**

In complete mix aeration the influent and the returned sludge are mixed and applied at several points along the length and width of the basin. The contents are mixed, and the mixed liquor suspended solids (MLSS) flows across the tank to the effluent channel. The oxygen demand and organic loading are uniform along the entire length of the basin.

**Contact Stabilization**

In contact stabilization, primary treatment is not required. The activated sludge is mixed with influent in the contact tank where the organics are absorbed by microorganisms. The MLSS is settled in the clarifier. The returned sludge is aerated in the reaeration basin to stabilize the organics. The process requires approximately 50% less tank volume and can be prefabricated as a package plant for flows of 0.05 to 1.0 MGD. On the downside, this system is more complicated to control because many common control calculations do not work.

**Extended Aeration Activated Sludge**

Extended aeration does not require primary treatment. It utilizes a large aeration basin where a high population of microorganisms is maintained. It is used for small flows from subdivisions, schools, etc. Prefabricated package plants utilize this process extensively. It ha a channel inthe shape of a race track, with rotors being used to supply oxygen and maintain circulation. Typically the process produces high-quality effluent and less activated sludge. (Oxidation ditch is a variation of extended aeration process).

**Pure Oxygen Systems**

Oxygen is diffused into covered aeration tanks. A portion of gas is wasted from the tank to reduce the concentration of carbon dioxide. The process is suitable for high-strength wastes where space may be limited. Special equipment for generation of oxygen is needed.

**Operation**

Operation of the activated sludge process requires more operator control than the other treatment processes discussed. The operator must adjust aeration, return rates and waste rates to maintain the balance of food, organisms and oxygen. Operators must observe operation of the aeration basin to check on mixing pattern, type and amount of foam (normally small amounts of crisp white foam), color of activated sludge (normally dark, chocolate brown), and odors (normally musty or earth odor). In regard to the settling tank, observations include flow pattern (normally uniform distribution), settling, amount and type of solids leaving with the process effluent (normally very clean).

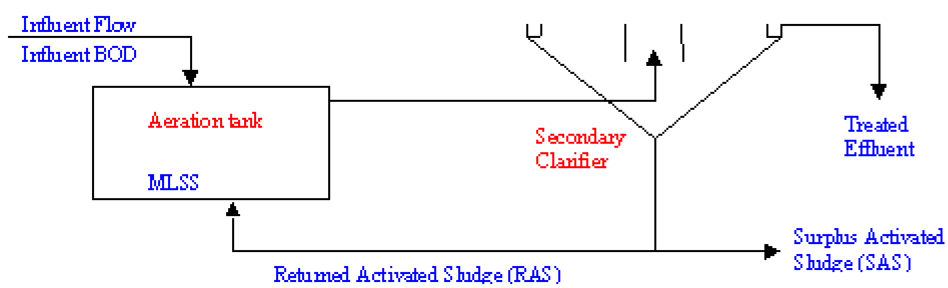
In process control operations, sampling and testing are important. Testing may include settleability testing to determine the settled sludge volume; suspended solids testing to determine influent and mixed liquor suspended solids, return activated sludge solids, and waste activated sludge concentrations; determination of the volatile content of the mixed liquor suspended solids; dissolved oxygen and pH of the aeration tank; BOD and/or COD of the aeration tank influent and process effluent; and microscopic evaluation of the activated sludge to determine the predominant organism.

## F/M Ratio

**Food- Microorganism Ratio**

One of the most fundamental control parameters for the activated sludge process is the relationship between the load (i.e. kg/day as opposed to mg/l) of BOD (or bacterial 'food') entering the aeration plant, and the 'mass' of bacteria in the aeration tank available to treat the incoming BOD. This is therefore known as the Food to Mass ratio (F: M ratio), also often referred to as the Sludge Loading Rate (SLR).

The amount of biomass within the reactor is known as the Mixed Liquor Suspended Solids (MLSS). Having established the MLSS by filtration and drying at 105 °C to constant weight, it is then possible to calculate the F: M ratio using the following simple model.




The number of microorganisms which are used to seed the aeration chamber is carefully controlled and is based on the **food to microorganism ratio** (**F/M ratio**). Microorganisms will most efficiently break down the organic matter in water if they are present in the right proportion. If the appropriate food to microorganism ratio is followed, then there will be efficient B.O.D. removal in the aerator. The best food to microorganism ratio is about 0.6.

Based on the food to microorganism ratio, some sludge is sent to the aeration basin from the clarifier while the rest of the sludge is sent to waste. The sludge re-circulation rate is controlled by a pump and valve combination. The valve can be opened or closed as needed in order to secure the optimum food to microorganism ratio.

**Calculating the F/M Ratio**

In order to calculate the proper amount of microorganisms to be added to the aeration basin, you will need to use the following formula:

$$\text{F/M ratio} = \frac{\text{food}}{\text{microbes}}$$

**Food - to - Microorganism Ratio**

$$F/M = \frac{S_o}{(\Theta)(X)}$$

$F/M$ : Food - to - Microorganism Ratio (1 / day)
$S_o$ : Influent BOD or COD Concentration ($g/m^3$)
$\Theta$ : Hydraulic Detention Time (day)
$X$ : MLVSS Concentration ($g/m^3$)

**Mean Cell - Residence Time**

**(Based on Aeration Tank Volume)**

$$\Theta_c = \frac{(V_r)(X)}{(Q_w)(X_w) + (Q_e)(X_e)}$$

$\Theta_c$ : Mean Cell - Residence Time (day)
$V_r$ : Aeration Tank Volume ($m^3$)
$X$ : MLVSS Concentration ($g/m^3$)
$Q_w$: Waste Sludge Flowrate ($m^3/day$)
$X_w$: MLVSS Concentration in the Waste Sludge ($g/m^3$)
$Q_e$ : Treated Effluent Flowrate ($m^3/day$)
$X_e$ : MLVSS Concentration in the Effluent ($g/m^3$)

**(Based on Total System Volume)**

$$\Theta_c = \frac{X_t}{(Q_w)(X_w) + (Q_e)(X_e)}$$

$X_t$ : Total Mass of MLVSS in the System (g)

**Returned Activated Sludge**

Returned Activated Sludge or RAS has some typical properties. When dried, 100 grams of RAS weighs 5 grams. When cooked at 550°C, 100 grams of RAS weighs 3 grams. And, most importantly for our calculations, 2% of RAS is microbes. So, if you have 100 grams of RAS, 2 grams of this is made up of microbes.

**Sludge Volume Index**

Sludge volume index is a quality indicator. It reflects the settling quality of the sludge. As the SVI increases, the sludge settles slower, does not compact as well, and is likely to result in more effluent suspended solids.

$$SVI = \frac{\text{Settled Sludge Vol}_{30}\text{, mL/L} \times 1{,}000}{\text{Mixed Liquor Suspended Solids, mg/L}}$$

Sample problem :

The sample used in the previous example (SSV) has an MLSS concentration of 2,800 mg/L. What is the SVI?

$$SVI = \frac{550 \text{ mL/L} \times 1{,}000}{2{,}800 \text{ mg/L}} = 196.43$$

**Bulking of Sludge**

Bulking can be said to have occurred in activated sludge plants when the sludge does not settle easily and has an excessive volume. This can lead to carry over from the final effluent clarifies. A bulking sludge is usually characterised by a sedimentation rate of less than 0.3 m/h, an SSVI or SVI of above 120 and 180 ml/g respectively and a low density structure.

The cause of bulking is normally due to either excessive filamentous growth or micro-organisms producing extracellular such as polysaccharide.

There are a number of short-term control measures which include biocide addition, use of flocculating chemicals or increasing the RAS flow. These measures generally treat the symptoms and not the underlying cause of the problem. The production of extracellular material and excessive filamentous micro-organisms have been related to nutrient deficiency, low dissolved oxygen and configuration of the aeration basin. This article deals with the control of activated sludge bulking based on these causes.

**Nutrient Deficiency**

A bulking sludge with a nutrient deficiency can be identified via a simple wastewater analysis of the influent wastewater and comparison of the BOD, N and P concentrations. Ideally the ratio of BOD:N:P should be 100:5:1. The limiting nutrients are generally N and P. This ratio however is the maximum requirement of most plant and many plants operate successfully at lower ratios. A bulking sludge with a nutrient deficiency is often viscous due to excessive production of extracellular polysaccharide and may also contain filamentous types 021N and *Thiothrix*.

Factors within the aeration tank can effect the concentration and requirement of nutrient dosing. Firstly, a decrease in temperature results in more nutrients being needed for carboneous removal as the BOD load is used for cell maintenance. Secondly, high sludge age results in lower nutrient requirements due to cell lysis. Thirdly, nutrient dosing may be required in instances were the BOD is readily available (eg it is present as a simple sugar) but the N and P are organically bound and are therefore not available for utilisation at a sufficiently high rate. Fourthly, when treating a variable carbonaceous loads with a tendency to nutrient deficiency the nutrient should be dosed continually to reduce the impact of shock loads. Yet it is possible to overdose N if only the concentration of ammonia is monitored in the effluent and nitrification occurs.

In general a soluble inorganic N and P concentration of 0.5 to 1 mg/l should be maintained within the aeration tank. However where a highly soluble BOD load is being treated the N and P minimum concentration may have to be increased to 1 to 3 mg/l. Some plants use commercial fertiliser which contains urea, ammonia and nitrate. However in these mixes the nitrate can lead to denitrification, therefore formulas without nitrate should be used.

**Low Dissolved Oxygen Concentrations**

A high F:M ratio requires a high DO concentration for effective treatment of the BOD load. However at low DO concentrations and at high F:M ratios excessive filamentous growth may occur leading to bulking. The filamentous micro-organism associated with low DO are *S. natans*, type 1701, *M. parvicella*and possibly *H. hydrossis.*

Bulking due to low DO can occur in a short space of time. A correction mechanism could be to increase the DO in the aeration tank. However this could lead to nitrification due to excess aeration and also for washout of the filamentous micro-organisms three sludge ages are required. Alternatively the F:M ratio could be decreased which would mean an increase in MLSS concentration. An increase in the MLSS concentration may lead to nitrification, due to increased sludge ages, or may exceed the secondary clarifiers design load. There are only really two solutions to this problem to either use a biocide or modify the aeration basin.

**Aeration Basin Design**

Activated sludge plants, which operate under complete mix system, produce poor settling sludge than systems which incorporate a selector box. In such systems a high substrate concentration zone is artificially created by mixing the RAS with the influent wastewater. This favours the growth of the floc-forming bacteria over filamentous micro-organisms. Floc-forming micro-organisms are favoured due to their high bioadsorption ability which limits the substrate available for other micro-organisms. However if the bioadsorption ability of the RAS has not been regenerated before contact with the influent then the efficiency of this process will be reduced.

Selector boxes can be either anoxic, aerobic or anaerobic. Anoxic zones are primarily used at nitrifying plants to remove nitrate and are effective due to floc-forming micro-organisms ability to denitrify and bioadsorb at high rates. An aerobic selector box should be based on a design F:M of 12 kg COD/kg MLSS, while anoxic selector boxes design load should be 1.2 kg COD/kg MLSS both with a retention time of 20 minutes. Anaerobic selector boxes have a similar F:M load rate to anoxic selector boxes but the retention time is between 45 minutes and two hours.

The above strategies provide starting points for activated sludge bulking control. However to understand the cause of bulking on an activated sludge plant the causative micro-organisms and the operating conditions which give rise to bulking should be first identified.

**Typical Design Parameters for Activated Sludge Process Modifications**

| Process Modification | SRT (day) | F:M ($kg\ BOD_5/kg\ MLVSS.d$) | L ($kg\ BOD_5/m^3.d$) | MLSS (mg/L) | t (hour) | R (%) |
|---|---|---|---|---|---|---|
| Conventional | 5 - 15 | 0.2 - 0.4 | 0.03 - 0.06 | 1.5 - 3.0 | 4 - 8 | 25-75 |
| Complete Mix | 5 - 15 | 0.2 - 0.6 | 0.07 - 0.18 | 2.5 - 4.0 | 3 - 5 | 25-100 |
| Step Feed | 5 - 15 | 0.2 - 0.4 | 0.06 - 0.09 | 2.0 - 3.5 | 3 - 5 | 25-75 |
| Modified Aeration | 0.2 - 0.5 | 1.5 - 5.0 | 0.12 - 0.23 | 0.2 - 1.0 | 1 - 3 | 5-25 |
| Contact-Stabilization | 5 - 15 | 0.2 - 0.6 | 0.09 - 0.12 | 1.0 - 3.0 | 0.5-1 | 50-150 |
| Stabilization | | | | 4.0-10.0 | 3 - 6 | |
| Extended Aeration | 20 - 30 | 0.05 - 0.15 | 0.01 - 0.04 | 3.0 - 6.0 | 18-36 | 50-150 |
| High Rate Aeration | 5 - 10 | 0.4 - 1.5 | 0.15 - 16.00 | 4.0-10.0 | 2 - 4 | 100-500 |
| High Purity Oxygen | 3 - 10 | 0.2 - 1.0 | 0.15 - 0.30 | 2.0 - 5.0 | 1 - 3 | 25-50 |
| Oxidation Ditch | 10 - 30 | 0.05 - 0.30 | 0.01 - 0.05 | 3.0 - 6.0 | 8-36 | 75-150 |

## Design of ASP

### Design Consideration

The items for consideration in the design of activated sludge plant are aeration tank capacity anc recycle and excess sludge wasting.

### Aeration Tank

The **volume of aeration tank** is calculated for the selected value of $\theta_c$ by assuming a suitable value of MLSS concentration, X.

$$VX = \frac{YQ\theta_c(S_O - S)}{1 + k_d\theta_c}$$

Alternately, the tank capacity may be designed from

$F/M = QS_O / XV$

Hence, the **first step** in designing is to choose a suitable value of $\theta_c$ ***(or F/M)*** which depends on the expected winter temperature of mixed liquor, the type of reactor, expected settling characteristics of the sludge and the nitrification required. The choice generally lies between 5 days in warmer climates to 10 days in temperate ones where nitrification is desired along with good BOD removal, and complete mixing systems are employed.

The **second step** is to select two interrelated parameters ***HRT, t and MLSS concentration***. It is seen that economy in reactor volume can be achieved by assuming a large value of X. However, it is seldom taken to be more than 5000 $g/m^3$. For typical domestic sewage, the MLSS value of 2000-3000 mg/l if conventional plug flow type aeration system is provided, or 3000-5000 mg/l for completely mixed types. Considerations which govern the upper limit are: initial and running cost of sludge recirculation system to maintain a high value of MLSS, limitations of oxygen transfer equipment to supply oxygen at required rate in small reactor volume, increased solids loading on secondary clarifier which may necessitate a larger surface area, design criteria for the tank and minimum HRT for the aeration tank.

The **length** of the tank depends upon the type of activated sludge plant. Except in the case of extended aeration plants and completely mixed plants, the aeration tanks are designed as long narrow channels. The **width** and **depth** of the aeration tank depends on the type of aeration equipment employed. The depth control the aeration efficiency and usually ranges from 3 to 4.5 m. The width controls the mixing and is usually kept between 5 to 10 m. **Width-depth ratio** should be adjusted to be between 1.2 to 2.2. The length should not be less than 30 or not ordinarily longer than 100 m.

**<u>Oxygen Requirements</u>**

Oxygen is reqiured in the activated sludge process for the oxidation of a part of the influent organic matter and also for the endogenous respiration of the micro-organisms in the system. The total oxygen requirement of the process may be formulated as follows:

$$O_2 \text{ required (g/d)} = \frac{Q(S_O - S)}{f} - 1.42\, Q_w X_r$$

where, f = ratio of $BOD_5$ to ultimate BOD and 1.42 = oxygen demand of biomass (g/g)

The formula does not allow for nitrification but allows only for carbonaceous BOD removal.

**<u>Aeration Facilities</u>**

The aeration facilities of the activated sludge plant are designed to provide the calculated oxygen demand of the wastewater against a specific level of dissolved oxygen in the wastewater.

**<u>Secondary Settling</u>**

Secondary settling tanks, which receive the biologically treated flow undergo zone or compression settling. ***Zone settling*** occurs beyond a certain concentration when the particles are close enough together that interparticulate forces may hold the particles fixed relative to one another so that the whole mass tends to settle as a single layer or "blanket" of sludge. The rate at which a sludge blanket settles can be determined by timing its position in a settling column test whose results can be plotted as shown in figure.

***Compression settling*** may occur at the bottom of a tank if particles are in such a concentration as to be in physical contact with one another. The weight of particles is partly supported by the lower layers of particles, leading to progressively greater compression with depth and thickening of sludge. From the settling column test, the limiting solids flux required to reach any desired underflow concentration can be estimated, from which the rquired tank area can be computed.

The solids load on the clarifier is estimated in terms of (Q+R)X, while the overflow rate or surface loading is estimated in terms of flow Q only (not Q+R) since the quantity R is withdrawn from the bottom and does not contribute to the overflow from the tank. The secondary settling tank is particularly sensitive to fluctuations in flow rate and on this account it is recommended that the units be designed not only for average overflow rate but also for peak overflow rates. Beyond an MLSS concentration of 2000 mg/l the clarifier design is often controlled by the solids loading rate rather than the overflow rate.

## Design of Completely Mixed Activated Sludge System

**Problem:**

Design a completely mixed activated sludge system to serve 60000 people that will give a final effluent that is nitrified and has 5-day BOD not exceeding 25 mg/l. The following design data is available.

Sewage flow = 150 l/person-day = 9000 $m^3$/day
$BOD_5$ = 54 g/person-day = 360 mg/l ; $BOD_u$ = 1.47 $BOD_5$
Total kjeldahl nitrogen (TKN) = 8 g/person-day = 53 mg/l
Phosphorus = 2 g/person-day = 13.3 mg/l
Winter temperature in aeration tank = 18°C
Yield coefficient Y = 0.6 ; Decay constant $K_d$ = 0.07 per day ; Specific substrate utilization rate = $(0.038\ \text{mg/l})^{-1}\ (\text{h})^{-1}$ at 18°C
Assume 30% raw $BOD_5$ is removed in primary sedimentation, and $BOD_5$ going to aeration is, therefore, 252 mg/l (0.7 x 360 mg/l).

**Design:**

**(a) *Selection of $q_c$, t and MLSS concentration*:**

Considering the operating temperature and the desire to have nitrification and good sludge settling characteristics, adopt $q_c$ = 5d. As there is no special fear of toxic inflows, the HRT, t may be kept between 3-4 h, and MLSS = 4000 mg/l.

**(b) *Effluent $BOD_5$:***

Substrate concentration, $S = \frac{1}{qY}(1/q_c + k_d) = \frac{1}{(0.038)(0.6)}(1/5 + 0.07)$

S = 12 mg/l.

Assume suspended solids (SS) in effluent = 20 mg/l and VSS/SS =0.8.

If degradable fraction of volatile suspended solids (VSS) =0.7 (check later), $BOD_5$ of VSS in effluent = 0.7(0.8x20) = 11mg/l.

Thus, total effluent $BOD_5$ = 12 + 11 = 23 mg/l (acceptable).

**(c) *Aeration Tank:***

$VX = \frac{YQq_c(S_O - S)}{1 + k_d q_c}$ where X = 0.8(4000) = 3200 mg/l

or $3200\ V = \frac{(0.6)(5)(9000)(252-12)}{[1 + (0.07)(5)]}$

$V = 1500\ m^3$

Detention time, $t = \frac{1500 \times 24}{9000} = 4h$

$F/M = \frac{(252-12)(9000)}{(3200)(1500)} = 0.45$ kg $BOD_5$ per kg MLSS per day

Let the aeration tank be in the form of four square shaped compartments operated in two parallel rows, each with two cells measuring 11m x 11m x 3.1m

**(d) *Return Sludge Pumping:***

If suspended solids concentration of return flow is 1% = 10,000 mg/l

$R = \frac{MLSS}{(10000)-MLSS} = 0.67$

$Q_r = 0.67 \times 9000 = 6000\ m^3/d$

**(e) *Surplus Sludge Production:***

Net VSS produced $Q_wX_r = \frac{VX}{q_c} = \frac{(3200)(1500)}{(5)}(10^3/10^6) = 960$ kg/d

or SS produced =960/0.8 = 1200 kg/d

If SS are removed as underflow with solids concentration 1% and assuming specific gravity of sludge as 1.0,

Liquid sludge to be removed = 1200 x 100/1 = 120,000 kg/d
= 120 $m^3$/d

**(f) *Oxygen Requirement:***

1. For carbonaceous demand,
   oxygen required = ($BOD_u$ removed) - ($BOD_u$ of solids leaving)
   = 1.47 (2160 kg/d) - 1.42 (960 kg/d)
   = 72.5 kg/h
2. For nitrification,
   oxygen required = 4.33 (TKN oxidized, kg/d)
   Incoming TKN at 8.0 g/ person-day = 480 kg/day. Assume 30% is removed in primary sedimentation and the balance 336 kg/day is oxidized to nitrates. Thus, oxygen required
   = 4.33 x 336 = 1455 kg/day = 60.6 kg/h
3. Total oxygen required
   = 72.5 + 60.6 = 133 kg/h = 1.0 kg/kg of $BOD_u$ removed.

Oxygen uptake rate per unit tank volume = 133/1500
= 90.6 mg/h/l tank volume

**(g) *Power Requirement:***

Assume oxygenation capacity of aerators at field conditions is only 70% of the capacity at standard conditions and mechanical aerators are capable of giving 2 kg oyxgen per kWh at standard conditions.

Power required = $\frac{136}{0.7 \times 2}$ = 97 kW (130 hp)

= (97 x 24 x 365) / 60,000 = 14.2 kWh/year/person

**Problem:**

Design a conventional activated sludge process to treat domestic sewage with diffused air aeration system from the following data:

| | |
|---|---|
| Population | : 1, 50,000 |
| Sewage contribution | : 150 LPCD |
| Settled sewage $BOD_5$ | : 200 mg / l |
| Effluent $BOD_5$ required | : 18 mg / l |
| F/M ratio | : 0.2 |
| MLSS | : 3000 mg / l |
| Air requirement | : 100 $m^3$ / day per kg of $BOD_5$ removed. |

**Answer:**

**1. Influent Flow and Process Efficiency Required:**

$$\text{Sewage flow} = 1{,}50{,}000 \times 150 = 22.5 \times 10^6 \text{ l/d} = \frac{22.5 \times 10^6}{1000} = 22500 \text{ m}^3/\text{d}$$

$$\textbf{Efficiency required} = \frac{(200 - 18)}{200} \times 100 = \mathbf{91\%}$$

**2. Determination of Volume of Aeration Tank:**

F / M ratio = 0.2

MLSS = 3000 mg / l

$$\frac{F}{M} = \frac{Q\,S_o}{V\,X}$$

$$0.2 = \frac{22500 \times 200}{V \times 3000}$$

**Volume of aeration tank, V = 7500 $m^3$**

**3. Check for Hydraulic Retention Time (HRT):**

$$HRT = \frac{V}{Q} = \frac{7500 \text{ m}^3}{22{,}500 \text{ m}^3/\text{d}} = 0.333 \text{ d} = (0.3333 \times 24) \text{ hr}$$

**HRT = 8 hrs (Range 4 – 8 hrs) Safe**

**4. Check for Volumetric Loading:**

Volumetric loading = $Q\,S_o / V$

$$= \frac{22500 \text{ m}^3/\text{d} \times 200 \text{ mg/l}}{7500 \text{ m}^3} = 600 \text{ mg } BOD_5/\text{l}$$

**Volumetric loading = 0.6 Kg $BOD_5$ / $m^3$ (Range 0.3 – 0.7) Safe**

**5. Return Sludge Ratio:**

$$r = \frac{Q_r}{Q} = \frac{X}{(106 / SVI) - X}$$

Taking SVI = 100, we get

$$r = \frac{3000}{(10^6 / 100) - 3000} = 0.429$$

**Return Sludge Ratio, r = 42.9 %**

**6. Tank Dimensions:**

Adopt a depth of 3 m and width of 4.5 m,
Therefore, Length of aeration tank = 7500 / (3 x 4.5) = 555.55 m
Provide a continuous channel with 6 baffles, so as to get 7 sections, the length of each section being = 555.55 / 7 = 79.36 say 80 m.
Let the thickness of each baffle be 0.25 m.
Therefore, total width of tank = ( 7 x 4.5 ) + ( 6 x 0.25 ) = 33 m.
Hence **overall inner dimension of the tank are 80 m X 33 m X 3.5 m.**

**7. Check for Horizontal Velocity:**

$$Q + Q_r = \frac{22{,}500 + 0.429 \times 22{,}500}{24 \times 60} = 22.33 \ m^3 / min$$

Therefore, **Horizontal velocity = 22.33 / (3 x 5) = 1.654 m / min (safe)**

**8. Air Requirement:**

Air needed = 100 $m^3$ / d per kg of $BOD_5$ removed

$$= \frac{100 (200 - 18) \times 22.5 \times 10^6}{24 \times 60 \times 10^6}$$

**Air requirement = 284.375 $m^3$ / min**

**<u>Problem:</u>**

Design the aeration tank of a conventional ASP for treating domestic sewage. Use the following data:

i. Average flow = 15 X $10^6$ liters/day
ii. Average $BOD_5$ = 250 mg/l
iii. Average $BOD_5$ of the treated effluent = 30 mg/l

The town is situated around Bangalore.

**Answer:**

30% removal of BOD in primary tmt = 75 mg/l
BOD to be removed in ASP = (250 - 75) -30 = 145 mg/l
Detention time, T = 6.25 hrs
Capacity of aeration tank, C = 4000 $m^3$
Assume return sludge of 20%, Total capacity = 5000 $m^3$

# Unit 8

# Anaerobic Sludge Digestion

## Sludge Treatment Process

The end products of the treatment process are effluent and sludge. Sludge may be primary or secondary depending upon the whether the treatment was primary or secondary. Raw sludge or primary sludge is usually odourous and contains high amount of organic matter in it. Whereas secondary sludge is also organic but little less objectionable. Activated sludge is the end product of activated sludge process. Sludge is usually semi solid I nature which contains 0.25 to 12 percent of solids and consists about 96 to 98% of moisture content in it. If moisture content of sludge is decreased, its volume decreases and it will be easy to handle and dispose. The sludge treatment process thus includes thickening of the sludge, sludge digestion, conditioning, dewatering, drying and incineration.

Thickening reduces the moisture content of the sludge. Digestion is to decompose the organic matter present in the sludge both aerobically and anaerobically. Conditioning improves the drainability of digested sludge. The purpose of dewatering is to further reduce the volume of the sludge and thereby increasing the solids concentration. Drying reduces much more moisture content and enables the sludge to be ready for incineration. Incineration involves the combustion of the sludge in a reactor under high temperature to destroy the organic material. Ultimately the dewatered sludge or ash from incinerators is disposed of on land or in sea.

**Sludge Digestion Process**

Sludge digestion involves the treatment of highly concentrated organic wastes in the absence of oxygen by anaerobic bacteria. The stabilization of sludge by decomposing the organic matter under controlled anaerobic conditions is called as ‗sludge digestion'. During this process of sludge digestion, sludge gets broken into three forms- 1) digested sludge 2) supernatant liquid 3) gases of decomposition. The digested sludge is also called as humus and it is black in colour and has less moisture and thus less volume. It will be free from pathogenic bacteria but may contain cysts and eggs of protozoa and helminths.

The supernatant liquor will have liquefied and finely divided solid matter with a high rate of BOD. Many odourous gases like methane, carbon dioxide, hydrogen sulphide are emitted from the process of sludge digestion.

The anaerobic treatment of organic wastes resulting in the production of carbon dioxide and methane, involves three distinct stages. In the first stage, referred to as "acid fermentation", complex waste components, including fats, proteins, and polysaccharides are first hydrolyzed by a heterogeneous group of facultative and anaerobic bacteria. This action of bacteria starts fermentation and the end products of this process will be acid carbonates and volatile organic acids and gases like methane, carbon dioxide and hydrogen sulphide. The pH value falls down less than 6. The second stage or the Acid Regression Stage is an intermediate stage where the volatile organic acids and the nitrogenous compounds of the first stage are attacked by the bacteria to form acid carbonates and ammonia compounds. The decomposed sludge emits offensive odour and its pH value rises to 6.8. The main feature of this stage is that the decomposed sludge and the entrapped gases of decomposition becomes foamy and rises to the surface of the digester forming scum. This stage continues for about 3 months or so and the amount of BOD will be very high.

However in the third stage, referred to as "Alkaline fermentation", proteins and organic acids are decomposed by anaerobic bacteria into simple substances like ammonia, organic acids and gases. The digested sludge, which is alkaline in nature (with a pH of 7.5), is formed in this stage and it gets separated out from the liquid. Large quantity of methane gas which has high calorific value is emitted from this stage. The sludge is also called as ripened sludge. BOD comes sown and this process takes almost one month to get over.

**Stages of anaerobic sludge digestion (Extra reading)**

In anaerobic digestion soft wet types of biomass are converted into biogas and digested state. It is a complicated process, requiring many types of bacteria to cooperate in series. The products of one type of bacteria are used as feedstock by the bacteria performing the next step in the chain. To envisage the process, the series of conversions is divided into four stages: hydrolysis, acidogenesis, acetogenesis and methanogenesis. The stages of anaerobic digestion process is depicted in figure.

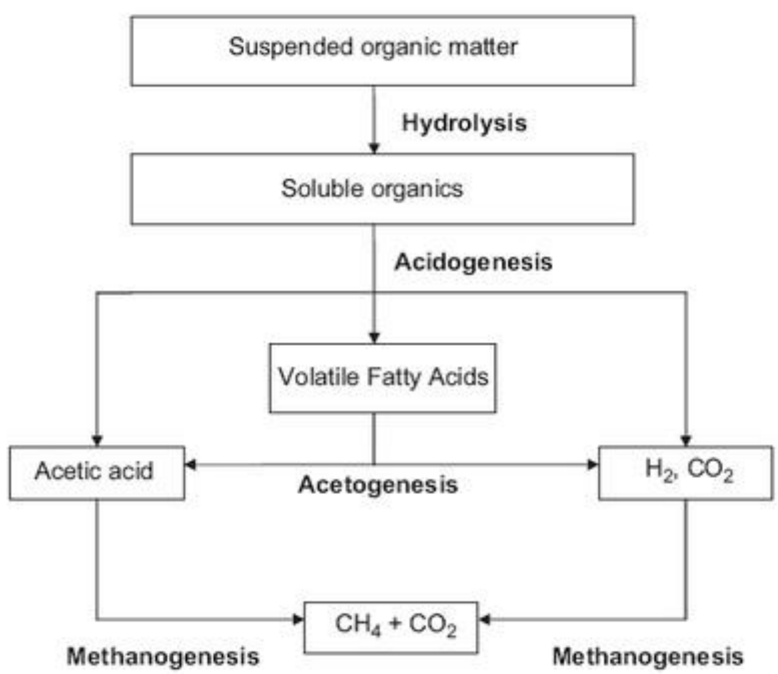



Figure : Stages/steps in the anaerobic digestion process

## Hydrolysis

The hydrolysis process transforms suspended organic matter into soluble organics. During the hydrolysis step, polymeric compounds are broken down by extra cellular enzymes into monomeric or dimeric compounds. The optimal pH for the hydrolysis step is 6, and the hydrolysis step usually is the rate limiting factor in anaerobic digestion.

A larger surface area enables enzymes to work faster because more enzymes are able to ‗attack' the organic material at the same time. Therefore a method to increase the rate of the hydrolysis is to increase the surface area of the substrate by previous grinding, boiling etc. Research is directed to apply this in practice. Also (bio)chemical pre-treatment of the organic material is being investigated, leading to more accessible polymers and hence to a higher rate and a higher degree of hydrolysis .

Also a two stage digester is used in practice, enabling the hydrolysis step to proceed under optimal conditions

## Acidogenesis

In the acidogenesis process the soluble organics that were produced by the hydrolysis are transformed into volatile fatty acids, mostly C2-C4 acids. In essence, glucose (sugar) is transformed into acids. While hydrolysis is the slowest process, acidogenesis is the quickest process.

The production of volatile fatty acids lowers the pH in the digester. When the pH drops below 4, the production of acids stops. It is therefore necessary that the following steps, the acetogenesis and methanogenesis take place at a sufficient rate. Otherwise the whole process stops, if the digester has turned acidic.

## Acetogenesis

In the acetogenesis process, the volatile fatty acids, (mostly) propionic acid, butyric acid and also ethanol are combined with water and are transformed into acetic acid, $CO_2$ and $H_2$.

## Methanogenesis

In this step the acetic acid, $CO_2$ and $H_2$ are converted into methane. In a stable digestion process around 70% of the methanogenesis converts acetic acid into $CH_4$ and $CO_2$, and this process is known as aceticlastic methanogenesis. This is the most efficient converstion from an energetic standpoint, as it produces the least amount of heat. The remaining 30% of the biogas is produced in the hydrogenotrophic methanogenesis, and this is the least effective energetic conversion. $H_2$ and $CO_2$ are converted into methane and water.

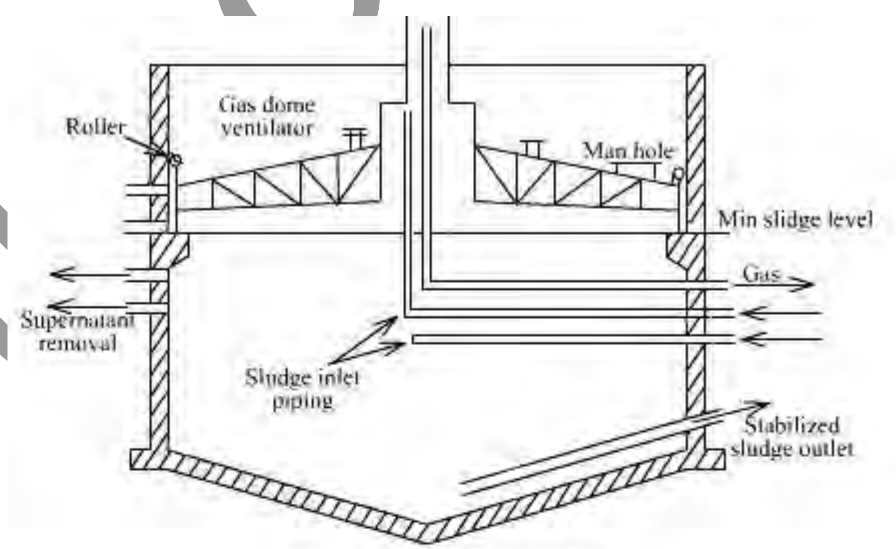
**Factors affecting process of Sludge Digestion**

The various factors which affect the process of sludge digestion are as follows:

1. *Temperature:* Rate of digestion is high at higher temperatures. There are two zone of temperature for digestion namely thermophilic zone and mesophilic zone. Thermophilic organisms or heat loving organisms which survive in temperature 40-60$^0$C are predominant in the first zone. Mesophilic organisms which survive in a temperature of 25 to 40$^0$C are predominant in the second.
2. *pH value:* A specific group of anaerobic bacteria called as methane formers convert the volatile acids to methane gas and if this bacteria does not work properly then the pH value of sludge drops to 5 and further bacterial action will be deteriorated. Therefore it is mandatory to keep the pH of the tank in the correct value. Acidity increases during sludge digestion because of certain other reasons like overdosing, over withdrawal and due to sudden admission of industrial waste.
3. *Seeding with the digested sludge:* During the starting of the operation of the tank, it is a must to seed it with digested sludge from other tanks to encourage the growth of useful bacteria. This will help in attaining proper balance conditions in the tank.
4. *Mixing and stirring of raw sludge with digested sludge:* the seeded sludge should be properly mixed with raw sludge for optimum conditions for the growth and activities of bacteria. This is achieved by stirring in a proper rate using mechanical stirring devices. The proper stirring results in even distribution of incoming sludge, breaks and reduces the scum and helps in increase of production of gases.

**Digestion Tanks or Digesters**

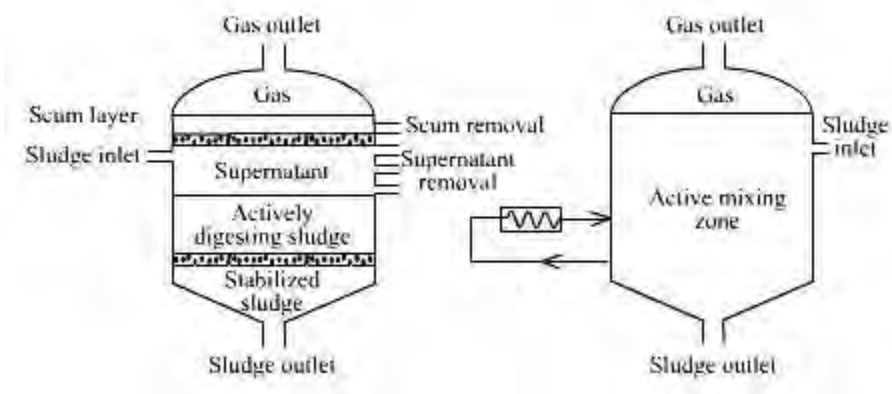
A sludge digestion tank is a RCC or steel tank of cylindrical shape with hopper bottom and is covered with fixed or floating type of roofs.




**Anaerobic sludge digester**

**Types of Anaerobic Digesters**

The anaerobic digesters are of two types: standard rate and high rate. In the standard rate digestion process, the digester contents are usually unheated and unmixed. The digestion period may vary from 30 to 60 d. In a high rate digestion process, the digester contents are heated and completely mixed. The required detention period is 10 to 20 d.




Standard-rate sludge digester

High-rate sludge digester

Often a combination of standard and high rate digestion is achieved in two-stage digestion. The second stage digester mainly separates the digested solids from the supernatant liquor: although additional digestion and gas recovery may also be achieved.

**Construction Details and Working of Sludge Digestion Tanks**

1. It is usually circular RCC tank with hoppered bottom and having fixed or floating type of roof.
2. The raw sludge is pumped into the digester and it is seeded with digested sludge from another tank.
3. Power driven mechanical devices or a screw pump is used to stir the sludge.
4. In cold countries digesters are provided with heating facility to maintain the optimum temperature.
5. The gases produced after decomposition is collected separately or in a gas dome in the same tank.
6. Digested sludge which gets settled in the bottom of the tank is removed periodically and the supernatant liquor is removed using withdrawal pipes kept at different heights.
7. The collected supernatant liquor is send back to the treatment plant for further treatment and the scum formed is broken by scum breakers or mechanical rakes.

**Design Considerations**

1. Cylindrical shaped tanks with dia ranging from 3-12m
2. Hopper bottom of slope 1:3
3. Generally digesters are designed to treat for a capacity upto 4 MLD.
4. Tank sizes are not less than 6 m diameter and not more than 55 m diameter.
5. Liquid depth may be 4.5 to 6 m and not greater than 9 m.
6. Capacity of digestion tanks, $V = \left[\frac{V_1 + V_2}{2}\right] \times t$

   Where V is the volume of the digester,
   $V_1$ is the raw sludge added per day (in $m^3/d$ ) and
   $V_2$ is the equivalent digested sludge produced per day on completion of digestion (in $m^3/d$ )
   t is the digestion period ( in days)
   Separate capacity is required during rainy seasons called as monsoon storage and therefore the total capacity of the tank is given by the formula
   $V = \left[\frac{V_1 + V_2}{2}\right] \times t + V_2 . T$
   Where T is the number of days which the digested sludge is stored.

The digester capacity may also be determined from this relationship

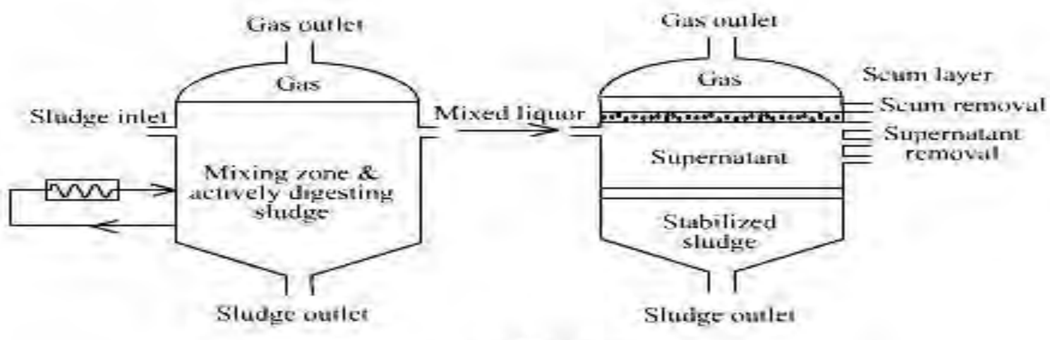
$$V = [V_f - 2/3\ (V_f - V_d)]t_1 + V_d t_2$$

where V = capacity of digester in $m^3$, $V_f$ = volume of fresh sludge $m^3/d$, $V_d$ = volume of daily digested sludge accumulation in tank $m^3/d$, $t_1$= digestion time in days required for digestion, d, and $t_2$ = period of digested sludge storage.

**Gas Collection**

The amount of sludge gas produced varies from 0.014 to 0.028 m3per capita. The sludge gas is normally composed of 65% methane and 30% carbondioxide and remaining 5% of nitrogen and other inert gases, with a calorific value of 5400 to 5850 kcal/m3.

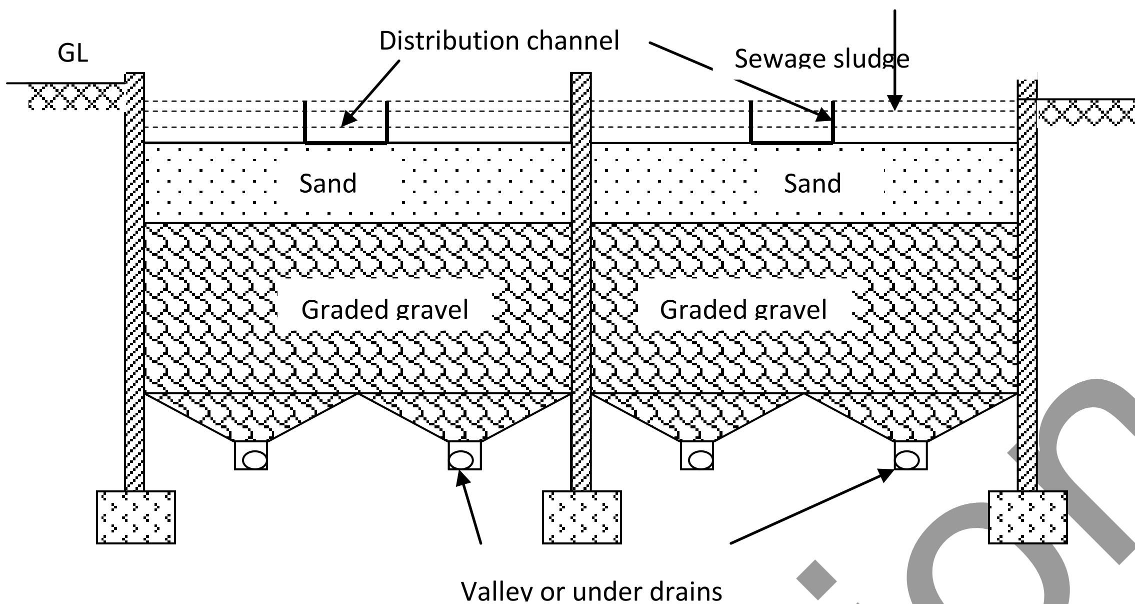
**Two stage Sludge Digestion**




**Two stage sludge digester**

**Sludge Drying Beds**

- The digested sludge contains lot of moisture content which should be eliminated before disposing it. It is usually achieved using sludge drying beds. The method consists of applying the sludge on specially prepared open beds of land. It consists of
- Open beds of land 45 to 60 cm deep.
- 30- 45cm thick graded layers of gravel or crushed stone in size varying from 15cm at bottom to 1.25 cm at top
- 10-25 cm thick coarse sand layer over the graded gravel layer
- Open jointed under-drain pipes of 15 cm in dia @ 5-7m c/c lay below the gravel layers at a slope of 1 in 100.
- 15 x 30 m in plan and are surrounded by brick walls rising about 1m above the sand surface.
- Top of the bed can be covered with glass to protect it from rains
- New sludge is spread only when the previous one has been removed.
- The sludge is spread on the top of the bed and a part of the moisture gets drain through the bed and some gets evaporated in the atmosphere. It takes 2 weeks to 2 months for complete drying. Usually sludge will be removed after 7-10 days, as within this period about 30% of the moisture gets away.




## Low Cost Waste Treatment Method

Many low-cost methods are able to treat excreta and sewage so that it can be reused. Reducing pathogens, particularly human intestinal nematodes and fecal bacteria, is the most important step in treating human waste. The WHO‘s guideline limit for fecal coliform bacteria is 1000 per 100 milliliters and the limit of nematodes is to be no more than one egg per liter. Once these standards are met, human excreta can be reused as fertilizer or for aquaculture. To provide the technology at a low cost and ensure sustainability, the facilities must be constructed out of locally available materials, adhere to the land-use patterns of the community, and conform to the geotechnical demands of the area. Human excreta do not necessarily have to be waste products, but can be reused for agriculture or aquaculture. The desire of the community to reuse excreta will affect the choices and operation of a sanitation program. Sanitation programs cannot simply be transplanted, but must be molded to fit the needs of each community, and thus they rely on innovation.

DRY SANITATION METHODS: - Dry sanitation methods do not use water as a carrier; instead, excreta are broken down by anaerobic methods (i.e., decomposition or dehydration). In decomposition systems, bacteria, worms, and other organisms break down urine and feces. Dehydration methods separate urine and feces, and then scatter feces with ash, shredded leaves, or sawdust to absorb excess moisture and deodorize. The added material also improves the nitrogen content in the event that the feces are reused as fertilizer.

a) Decomposition Systems: Pit Latrines and Ventilated Improved Pit (VIP) Latrines -Pit latrines are the most rudimentary form of sanitation. Structures made out of locally available materials cover a defecation hole—a pit dug in the ground to collect waste. Once full, the pit is covered with sediment. The water table should be no less than 0.5 meters below the surface of the pit or it could contaminate the ground water. Geological conditions are a primary concern when considering a pit latrine; rocky substrates and shallow water tables negate this option for many communities, and areas with non-cohesive soils require a lined pit. The health problems posed by pit latrines have been widely documented. The open defecation hole attracts mosquitoes and flies and produces a ghastly odor. Pit latrines often serve as breeding grounds for mosquitoes, thus increasing the incidence of malaria in some areas. These adverse conditions lead many communities to abandon latrines.
b) Two pit Latrines or Ventilated Improved Pit (VIP) latrines are an improvement over traditional latrines in two important respects: they mitigate the noxious odor and reduce the number of flies and other insects that plague users of traditional latrines. In a VIP latrine, a vent pipe allows fresh air to flow through the latrine, reducing odor. The vent also allows light into the latrine, attracting insects into the pipe, where they are trapped by the fly screen at the top of the pipe. The screen also keeps out insects looking to enter the pipe from the outside. The dry decomposition options utilizing anaerobic breakdown have been developed to allow excreta to be reused for agricultural purposes. If VIP latrines are constructed with two pits, instead of moving the latrine when the pit is full, users switch to the other pit. After the waste in the full pit composts, it can be reused as fertilizer. The amount of time before the compost can be used as fertilizer depends on climate and ranges from 3–12 months.
c) Dehydration Systems: - Dehydration systems separate urine and feces using a special pedestal or urine diversion pan. Urine is diverted into a holding pot or into a soak field, while a watertight vault collects the feces. After defecation, ash or another absorbent (e.g., lime, dry soil, husks, organic matter) is sprinkled into the vault. Material used for anal cleansing is put into another container rather than dropped into the vault. Once the vault is three-quarters full, the feces is covered with dry earth. Both the urine and the dehydrated feces can be reused as fertilizer. Urine is often used immediately, but it should ideally sit for six months to ensure that nematode eggs are destroyed. Dehydrated feces should not be used for at least a year, although case studies identify different amounts of storage time.

   One advantage of dehydration systems is better groundwater protection due to the use of watertight and aboveground vaults, which can be used in areas that have geotechnical limitations. The absorbent material also helps to deodorize the chamber and reduce flies. Dehydration can be employed in a wide range of climates. Due to the specific nature of the technology, however, the most common problem is moisture entering the dehydration chamber, either from leaks, urine splashing into the chamber, or other accidental spills.

Children might find the latrines more difficult to use, and blocked urine separators also pose problems.

Both dehydrated and composted human excreta can be used for many different purposes at the community and individual levels. By selling excreta for agricultural or aquacultural use, a community can recoup the costs of its initial investment in sanitation. Excreta can serve not only as a fertilizer, but also as a soil conditioner, due to its high organic content. Ponds using wastewater have been found to be productive, possessing high pH and oxygen levels; in addition, the fish are not susceptible to enteric bacteria Using excreta to grow duckweed, algae, and water hyacinth are other options; duckweed can be used in animal feed or fish food.

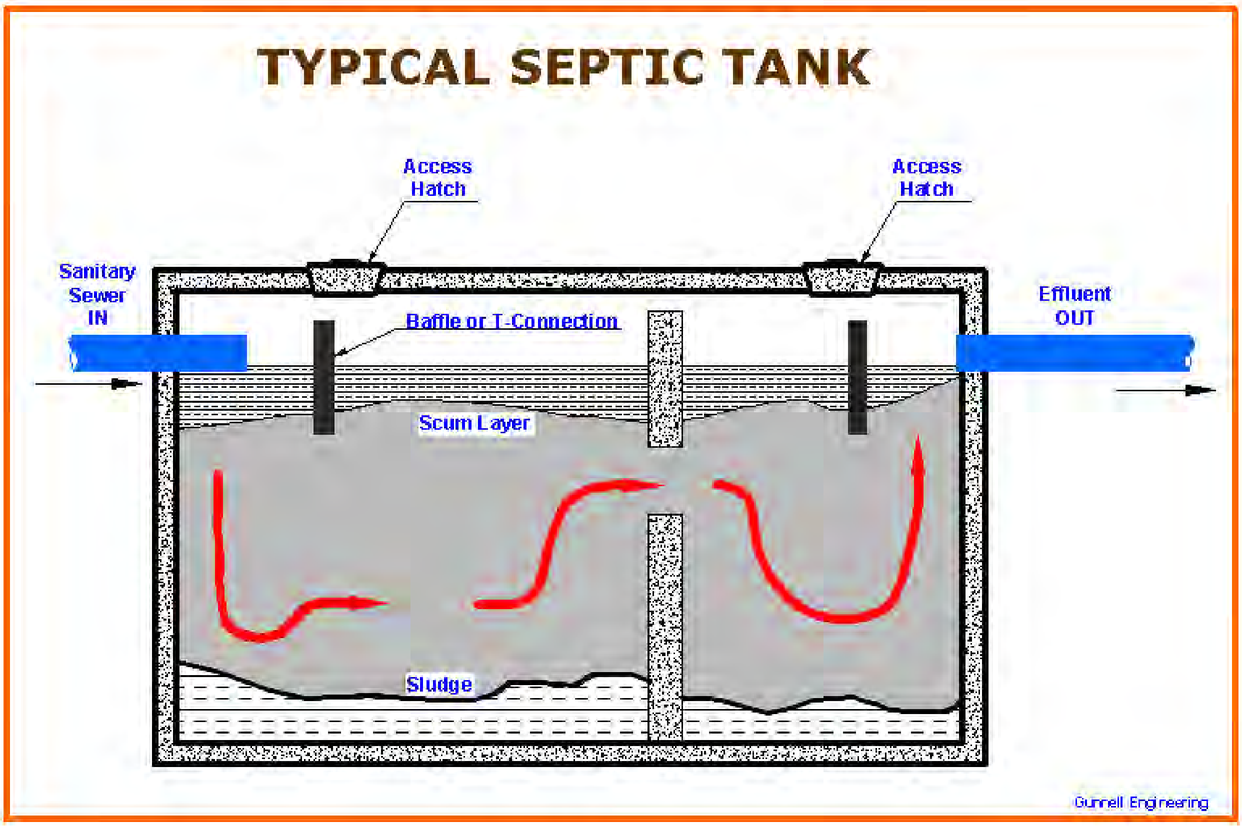
d) Biogas is another way to reuse human excreta—and provide a much-needed resource. The anaerobic decomposition of human excreta produces methane gas, which can be harnessed by biogas plants to produce energy. These plants can be designed to operate at the individual household level and produce tanks of biogas for domestic cooking and lighting. One person produces one cubic foot of biogas per day—enough to meet the daily energy needs of a person in the developing world

WET SANITATION METHODS: - Wet sanitation methods utilize water to treat waste. These methods are only recommended for communities that have liberal supplies of water. The most widely used models are the pour flush latrine, the aqua privy, and the septic tank. These systems are usually more expensive than the VIP latrine. Primary treatment produces effluent and sludge; ability to reuse the effluent depends on household land-use patterns.

a) Pour Flush Latrines: - A pour flush latrine consists of a cover slab and a special pan that provides a water seal. A U-shaped pipe is used to maintain the water seal. Approximately 1–3 liters of water are needed for each flush. The latrines can be constructed with pits directly underneath or offset, or with two pits. They can also be built inside a dwelling, with the pit located outside. If properly built and maintained, pour flush latrines reduce odors and flies. They should be considered in communities where anal cleansing habits require the use of water. Disadvantages of pour flush latrines include the high water requirements, higher cost, and problems caused by clogged pipes. The pour flush latrine is used in parts of Asia and the Caribbean, and most widely in India, where it is called the Sulabh toilet. The Sulabh toilet replaced the bucket system, saving more than 60,000 people (mostly women) from manually handling waste. In addition, public pour flush latrines connected to biogas plants generate electricity.

b) Aqua privy:-An aqua privy is an underground watertight tank, filled with water, which is connected to a flush toilet or defecation hole. The tank is located directly underneath the toilet and separates solid matter from liquids. The tank can also be used to dispose of greywater. Over time, the solid matter in the tank degrades anaerobically. A soak field absorbs the effluent; however, sludge must be removed from the tank every 1–5 years. Usually a vacuum tanker or service crew performs this difficult and potentially dangerous task. A bucket of water must be poured down the drop pipe daily to clear any buildup. If operated properly, there are usually no problems with flies or odors. The tank must be maintained; if the tank is leaking, odor can become a problem. The soak fields used by aquaprivies and septic tanks can also cause problems.

c) Septic Tank: -




A septic tank is similar to an aquaprivy, except that a septic tank can be located outside the house. The toilet used with a septic tank also has a U-trap water seal. As with the aquaprivies, septic tanks can be used to dispose of greywater and must be periodically emptied of sludge. They also require the use of a soak field for the secondary treatment of effluent. Septic tanks may have two chambers to separate and promote further settlement of liquid and solid excreta.

Septic tanks are more costly than aquaprivies; given the higher initial investment required, plus the recurring costs of emptying the tanks, this method is not generally recommended for poor rural communities. For peri-urban areas, the ability to connect the household to a sewage system at a later date is a major benefit. The disadvantages include faulty or leaking septic tanks, water requirements, higher costs, and the use of a soak field. If the septic tank is faulty, flooding can cause hydraulic overloading.

Soak fields or soak pits, also known as soil absorption systems, treat the effluent from aquaprivies and septic tanks. A soak field is comprised of drainage ditches or gravel-lined trenches that allow effluent to percolate through the soil, achieving secondary treatment by absorption and biodegradation. A conventional soil absorption system allows the effluent from a septic tank to outflow into perforated pipe laid in the bottom of trenches two-feet deep; stoneware can also substitute for pipe. The soak field presents health risks, as the effluent coming out of the tank could contain pathogens or nematode eggs. The effluent is potentially hazardous to humans and the area‘s groundwater. In addition, the effluent could overflow the trenches if it exceeds the absorptive capacity of the soil. The soak field also requires that the user possess an adequate amount of land with certain geological characteristics; septic tanks and soak fields cannot be located on a slope, in flood zones, or in areas with shallow water tables. In addition, areas with non-permeable soil do not allow the percolation necessary to achieve secondary treatment.

Other natural treatment processes of waste water treatment are phytoremediation: planting the trenches of a soak field with native semi-aquatic plants, flowers, or vegetables. This process ensures that the soak field maintains equilibrium and will not overflow; provides a safe conduit for effluent; and also produces end products that can be decorative.

A septic tank is a water tight single storied, underground, horizontal continuous flow sedimentation tank in which sewage is retained sufficiently long to permit 60-70% of suspended solids to settle in the form of sludge at the bottom of the tank. Some of the lighter solids including grease and fat rise to the surface of the sewage to form floating scum. The scum and the sludge so formed are then retained with the sewage in the tank for a several months during which they are decomposed by the process called sludge digestion. Consequently, there is a reduction in volume of the sludge to be disposed off. There will be release of gases like $CO_2$, $CH_4$ and $H_2S$. The effluent although clarified to some extent will still contain considerable amount of dissolved and suspended putrescible organic solids and viable pathogens and therefore disposal of effluent merits careful consideration. Because of the unsatisfactory quality of the effluent and also difficulty to providing a proper disposal system for the effluent, septic tanks are recommended only for individual homes and small communities and institutions whose contributory population does not exceed 300.

Design considerations

A properly designed septic tank should provide for the following:

a) Space for sewage retention to allow sedimentation.

**Design problem:**

Design a septic tank for a small colony of 200 persons provided with a water supply of 135 lpcd. Assume the necessary data for the design. The data available are: MDD is twice the ADD, 80% of water supply becomes spent, detention time of 24 hours, length to breadth ratio is 1:3. Draw a line diagram showing designed dimensions.

**Answer:**

$Q_{sewage}$ = (200 x 135 x 2) x 0.8 = 43, 200 l/d
Volume = (43,000/24) x 24 = 43.2 $m^3$
Total volume of sludge = 30 x 200 x 1% = 6 $m^3$
Total capacity of tank = 43.2 + 6 = 49.2 $m^3$
Assuming a depth of tank = 1.5 + 0.3 free board
Surface area, A = 49.2/1.5 = 32.8 $m^2$
Length = 3 x Breadth
A = L x B
B = $\sqrt{A/3}$
B = 3.31 say 3.3 m
L = 9.93 say 10 m
L x B x D = 10 m x 3.3 m x 1.8 m

## <u>Stabilization Ponds</u>

The stabilization pond is low cost waste water treatment method as the construction cost and operational cost are low and if land is available for a cheap price. The stabilization ponds are open flow through basins specifically designed and constructed to treat sewage and biodegradable industrial wastes. They provide long detention periods extending from a few to several days. The released effluent can be directly used for irrigation in semi arid other water scarce region. BOD, SS and pathogen removal is found to be excellent.

Pond systems, in which oxygen is provided through mechanical aeration rather than algal photosynthesis, are called aerated lagoons.

Lightly loaded ponds used as tertiary step in waste treatment for polishing of secondary effluents and removal of bacteria are called maturation ponds.

**Classification of Stabilization Ponds**

Stabilization ponds may be aerobic, anaerobic or facultative.

<u>Aerobic ponds</u> are shallow ponds with depth less than 0.5 m and BOD loading of 40-120 kg/ha.d so as to maximize penetration of light throughout the liquid depth. The stabilization of waste is brought about by aerobic bacteria. Such ponds develop intense algal growth. The oxygen demand of the bacteria in such ponds is provided by algal photosynthesis and this oxygen is

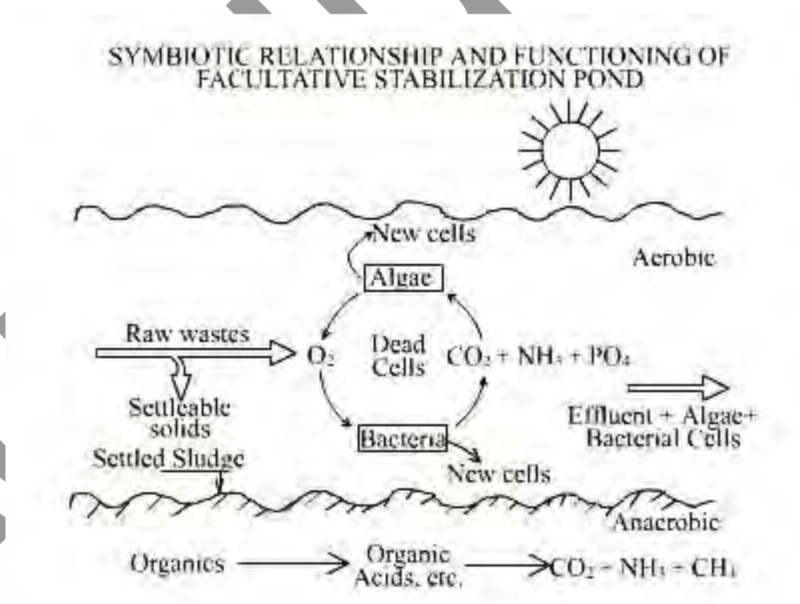
utilized by bacteria for oxidising waste organic matter. This process is called as ALGAL-SYMBIOSIS.

In anaerobic ponds , the waste stabilization is brought about by anaerobic bacteria and it converts organic matter into carbon dioxide, ammonia and phosphates. These ponds are used as pretreatment of high strength wastes with BOD load of 400-3000 kg/ha.d Such ponds are constructed with a depth of 2.5-5m as light penetration is unimportant.

Facultative pond functions aerobically at the surface while anaerobic conditions prevail at the bottom. They are often about 1 to 2 m in depth. The aerobic layer acts as a good check against odour evolution from the pond. Facultative stabilization ponds are known as Oxidation ponds.

**Mechanism of Purification**

The functioning of a facultative stabilization pond and symbiotic relationship in the pond are shown below. Sewage organics are stabilized by both aerobic and anaerobic reactions. In the top aerobic layer, where oxygen is supplied through algal photosynthesis, the non-settleable and dissolved organic matter is oxidized to CO2 and water. In addition, some of the end products of partial anaerobic decomposition such as volatile acids and alcohols, which may permeate to upper layers are also oxidized periodically. The settled sludge mass originating from raw waste and microbial synthesis in the aerobic layer and dissolved and suspended organics in the lower layers undergo stabilization through conversion to methane which escapes the pond in form of bubbles.




**Factors Affecting Pond Reactions**

Various factors affect pond design:

a) Wastewater characteristics and fluctuations.

b) Environmental factors (solar radiation, light, temperature)
c) Algal growth patterns and their diurnal and seasonal variation)
d) Bacterial growth patterns and decay rates.
e) Solids settlement, gasification, upward diffusion, sludge accumulation.

The depth of aerobic layer in a facultative pond is a function of solar radiation, waste characteristics, loading and temperature. As the organic loading is increased, oxygen production by algae falls short of the oxygen requirement and the depth of aerobic layer decreases. Further, there is a decrease in the photosynthetic activity of algae because of greater turbidity and inhibitory effect of higher concentration of organic matter.

Gasification of organic matter to methane is carried out in distinct steps of acid production by acid forming bacteria and acid utilization by methane bacteria. If the second step does not proceed satisfactorily, there is an accumulation of organic acids resulting in decrease of pH which would result in complete inhibition of methane bacteria. Two possible reasons for imbalance between activities of methane bacteria are: (1) the waste may contain inhibitory substances which would retard the activity of methane bacteria and not affect the activity of acid producers to the same extent. (2) The activity of methane bacteria decreases much more rapidly with fall in temperature as compared to the acid formers.

Thus, year round warm temperature and sunshine provide an ideal environment for operation of facultative ponds.

**Algal Growth and Oxygen Production**

- Algal growth converts solar energy to chemical energy in the organic form. Empirical studies have shown that generally about 6% of visible light energy can be converted to algal energy.
- The chemical energy contained in an algal cell averages 6000 calories per gram of algae.
- Depending on the sky clearance factor for an area, the average visible radiation received can be estimated as follows:
- Avg. radiation= Min. radiation + [(Max. radiation - Min. radiation)x sky clearance factor]
- Oxygen production occurs concurrently with algal production in accordance with following equation:

$$106CO_2 + 16NO_3 + HPO_4 + 122H_2O + 18H^+ \longrightarrow C_{106}H_{263}O_{110}N_{16}P_1 + 138O_2$$

- On weight basis, the oxygen production is 1.3 times the algal production.

**Design Criteria**

a) The length of the tank may be kept at about twice the width.
b) The depth may be kept between 1 to 1.5 m

c) A free board of about 1m may also be provided above a capacity corresponding to 20-30 day of detention period.

d) Areal Organic Loading: The permissible areal organic loading for the pond expressed as kg BOD/ha.d will depend on the minimum incidence of sunlight that can be expected at a location and also on the percentage of influent BOD that would have to be satisfied aerobically. The Bureau of Indian Standards has related the permissible loading to the latitude of the pond location to aerobically stabilize the organic matter and keep the pond odour free. The values are applicable to towns at sea levels and where sky is clear for nearly 75% of the days in a year. The suitable value of organic loading can be worked out to be 300-150 kg/hectare/day in tropical countries like India.

e) Detention Time: it is usually from 7 days to 42 days or so. It can be calculated using the formula

$$\mathbf{t\ (in\ days) = (1/K_D)\ log_{10}\ [L / (L-Y)]}$$

where L is the BOD of the effluent entering the pond

Y = the BOD removed (usually 90 to 95% of L)

$K_D = 0.1$ / day at $20^oC$

**Advantages of oxidation Ponds**

1. Suitable for hot climate, where 200 sunny days are expected per year
2. Best suited for small cities and towns where land is cheap
3. Capital cost is only 20 to 30% of conventional plant
4. Maintenance cost is minor
5. No skilled supervision is required
6. Flexible and will get upset due to sudden fluctuations in organic loading

**Disadvantages of oxidation Ponds**

1. Mosquito nuisance
2. Bad odours

**Design problem:**

Arrive at the dimensions of an oxidation pond to treat 1 MLD of sewage received from a suburb of a city, located in an area of hot climate, the BOD of sewage being 320 mg/lt.

**Answer:**

Total BOD load = $1 \times 10^6$ l/d x320mg/l = $320 \times 10^6$ mg/d = 320 kg/d

Assuming a BOD loading of 300 kg/hec/day for an area located in hot climate.

Pond area = Total BOD/BOD loading = 320 kg/d/ 300 kg/hec/day = 1.067 ha = 10667 $m^2$

Detention Time

Winter, $k_{15} = 0.25 \times 1.06^{15-20} = 0.187/d$

Summer, $k_{35} = 0.25 \times 1.06^{35-20} = 0.6/d$

Detention time, $T = k_t/k_{15}$

For BOD removal efficiency of 85% and dispersion factor of 0.5, $k_t = 3$

T = 3/0.187 = 16.04 days (winter)

T = 3/0.6 = 5 days (summer)

Pond Volume = Flow x Detention time = $1 \times 10^3\ m^3 \times 16.04 = 16040\ m^3$

Pond water depth = Pond volume/Pond area = 16040/10667 = 1.5 m

L = 2.5 B

A = L x B

$B = \sqrt{A/2.5} = \sqrt{10667/2.5}$
B = 65.32 say 65 m

L = 164.1 say 164 m

Overall dimension of the pond = 164 m x 65 m x (1.5 + 0.3 m Free board)

## Oxidation Ditches (Extended Aerated Lagoons)

The Oxidation Ditch (OD) is a modified form of the activated sludge system. Oxidation ditches are mechanical secondary treatment systems which are tolerant of variations in hydraulic and organic loads. Treatment of wastewater using an oxidation ditch is relatively similar to wastewater treatment in a packaged plant. But the oxidation ditch replaces the aeration basin and provides better sludge treatment. The ODs can be easily adjusted to meet most combinations of incoming sewage and effluent standards. This system achieves both high BOD reduction and some nutrient removal. The only pretreatment typically used in an oxidation ditch system is the bar screen. After passing through the bar screen, wastewater flows directly into the oxidation ditch.
The OD consists of a "ring or oval shaped channel" equipped with mechanical aeration devices. Activated sludge is added to the oxidation ditch so that the microorganisms will digest the B.O.D. in the water. This mixture of raw wastewater and returned sludge is known as mixed liquor.

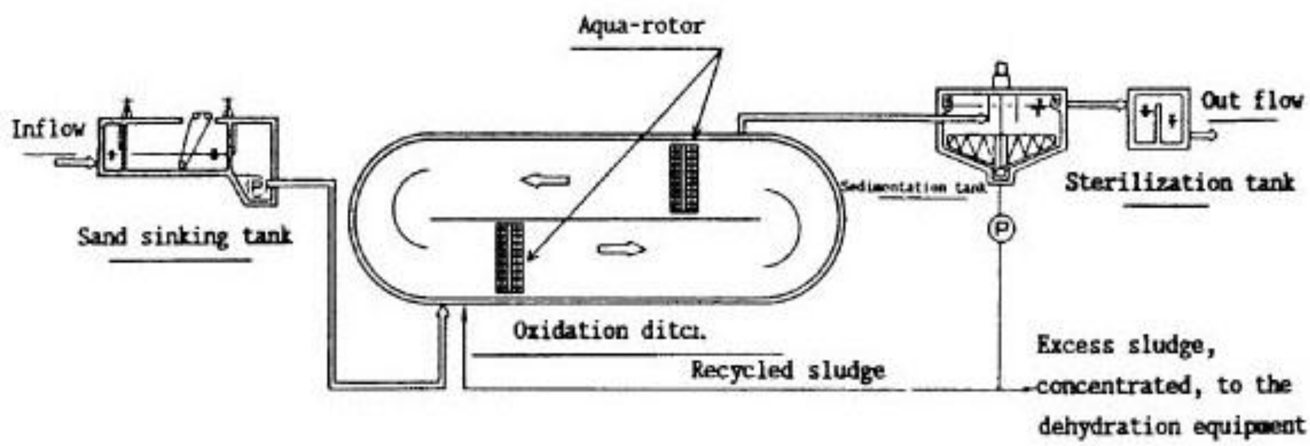
Screened wastewater, which enters the ditch is aerated and circulated. ODs typically have long detention times and are capable of removing between 75% and 95% of the Biological Oxygen Demand (BOD).

Oxygen is added to the mixed liquor in the oxidation ditch using rotating biological contactors (RBC's.) RBC's are more efficient than the aerators used in packaged plants. In addition to increasing the water's dissolved oxygen, RBC's also increase surface area and create waves and movement within the ditches Once the B.O.D. has been removed from the wastewater, the mixed liquor flows out of the oxidation ditch. Sludge is removed in the clarifier. This sludge is pumped to an aerobic digester where the sludge is thickened with the help of aerator pumps. This method greatly reduces the amount of sludge produced. Some of the sludge is returned to the oxidation ditch while the rest of the sludge is sent to waste.

The proprietary "Orbal System" uses three channels or ditches concentrically placed. Each channel is independently aerated and can be configured to act in parallel or series with the other channels, depending upon the degree of treatment required.

After screening and grit removal, sewage enters the outer channel where most of the biological reaction takes place. The second channel is held at a slightly higher dissolved oxygen content for further BOD and nutrient reduction. The innermost channel is used for polishing the effluent before it passes to a clarifier.

Typical figures for ODs are as follows:




Oxidation Ditch

**Advantages and Disadvantages**

The greatest advantage of an oxidation ditch is the efficiency of sludge removal. In an oxidation ditch, only about 15% of the original B.O.D. ends up as sludge, compared to a packaged plant where about 60% of the B.O.D. becomes sludge.

However, oxidation ditches are expensive to maintain. The monetary cost is very high per ton of B.O.D. removed. In some cases, the cost may reach nearly 350 dollars per ton.

Oxidation ditches have an additional environmental drawback. The water is moved through the ditches using rotors, and these rotors in turn use electricity. The electricity used to operate the plant causes sulphur dioxide and other contaminants to be released into the atmosphere from coal-burning electrical plants.

Oxidation ditches provide the most thorough process for treating sewage, but oxidation ditches are also one of the most costly forms of treatment.

**Design Criteria**

1. Detention period of 12 to 15 hours
2. Organic loading of 800 to 2500litres per kg of BOD
3. Concentration of suspended solids in the mixed liquir – about 4000 to 5000 mg/l
4. Volume of ditch – 120 to 150 cubic metre length of the roroe
5. Length of the standard horizontal axis cage type rotor used is 1m.
6. Rotor is in form of cylindrical cage of .7 m in dia and speed of 75RPM, while dipping 10cm into liquid.

Septic tanks and Imhoff tanks

For areas where water supply exist, but sewerage system is not yet installed, septic tanks and imhoff tanks are suitable. To deal with the sewage produce from such areas, septic tanks or Imhoff tanks are constructed.

## Reuse and Recycle of Waste water

Reclaimed water is an important component of wise water management. Reclaimed water is derived from domestic wastewater and small amounts of industrial process water or storm water. The process of reclaiming water, sometimes called water recycling or water reuse, involves a highly engineered, multi-step treatment process that speeds up nature's restoration of water quality. The process provides a high-level of disinfection and reliability to assure that only water meeting stringent requirements leaves the treatment facility.

Onsite wastewater management systems provide treatment and recycling/disposal of:

- greywater – shower, bath, hand basins, clothes washing machine, laundry troughs and the kitchen
- blackwater – toilet waste (water-flush, incineration or dry composting systems)
- sewage – combined greywater and blackwater

Wastewater treated to primary quality is only suitable for disposal below ground via soil absorption trenches, mounds and evapo-transpiration beds or trenches. Wastewater treated to secondary quality may also be dispersed to land via pressure-compensating subsurface irrigation. Greywater that has been treated to advanced-secondary quality can be used in the home for toilet flushing and cold water use in the clothes washing machine as well as surface and subsurface irrigation.



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Grey water is reusable wastewater from residential, commercial and industrial bathroom sinks, bath tub shower drains, and clothes washing equipment drains. It is reused onsite, typically for landscape irrigation. Use of non toxic and low-sodium soap and personal care products is required to protect vegetation when reusing grey water for irrigation.

Recycled water can satisfy most water demands, as long as it is adequately treated to ensure water quality appropriate for the use. In uses where there is a greater chance of human exposure to the water, more treatment is required. As for any water source that is not properly treated, health problems could arise from drinking or being exposed to recycled water if it contains disease-causing organisms or other contaminants. EPA regulates many aspects of wastewater treatment and drinking water quality.

**Advantages of Reuse &Recycling**

**1. Protects Environment:** The foremost benefit or recycling is that it helps in protecting the environment in the most balanced manner. While many trees are cut down continually, recycled paper made from certain trees is re-used repeatedly to minimize felling/ deforestation. With re-cycled paper as an outstanding example, a number of other natural resources can be reused this way.

**2. Reduces Energy Consumption:** A large amount of energy is consumed by processing raw materials at the time of manufacture. Recycling helps to minimize energy consumption, which is crucial for massive production, such mining or refining. This also makes the production process very cost-effective and beneficial for manufacturers.

**3. Reduces Pollution:** Industrial waste today is the main source of all types of pollution. Recycling of industrial products such as cans, chemical, plastics helps to cut down pollution levels considerably, as these materials are re-used, instead of throwing them away irresponsibly.

**4. Reduces Global Warming:** Recycling helps to alleviate global warming and its ill effects. Massive waste is burned in heaps which produces large amount of greenhouse gas emissions such as CO2 and CFC's. Recycling ensure that the burning process is minimized and any waste is re-generated as a useful product with no or minimal harmful impact on the environment. Recycling produces less greenhouse gases as industries burn fewer fossil fuels for eco-friendly products.

**5. Judicial and Sustainable use of Resources:** Recycling promotes judicial and sustainable use of resources. This process ensures that there is no discriminate use of any material when available in plenty in the present. Recycling is encouraged at all levels, starting from school to corporate offices and at international levels. This means we can preserve all precious resources for our future generation, without any compromise in the present.

**6. Conserves Natural Resources:** If old and used materials are not recycled, the new products are made from extracting fresh raw materials from beneath the earth through mining and extraction. Recycling helps in conserving important raw materials and protects natural habitats for the future. Conserving natural resources such as wood, water and minerals ensures its optimum use.

**7. Reduces Amount of Waste to Landfills:** Recycling old and waste products into new products reduces the amount of waste that go to landfills. This helps in reducing water and land pollution as landfills are a major source in contributing to destruction of natural environment. Recycling programs keep 70 tons of waste from being deposited into landfills every year.

**8. Create Green Jobs:** Recycling is good for the environment and apart from that it also creates green jobs. According to the U.S. Bureau of Labor Statistics, green goods and services accounted for 3.1 million jobs in the United States by 2010.

**Disadvantages of Reuse &Recycling**

**1. Not always Cost Effective:** Recycling is not always cost-effective. Sometimes, there may be a need to establish separate factories to process reusable products. This may create more pollution in terms of cleaning, storage and transportation.

**2. Recycled Products May not Last for Long:** Recycled products are always not of durable quality. Such items are mostly made of trashed waste, picked up from heaps other waste products which are of fragile or overly used. For this reason, recycled products are cheap and last for a shorter period.

**3. Unsafe and Unhygienic Recycling Sites:** Recycling sites are often unsafe and unhygienic. Places where all sorts of waste are dumped are conducive for debris formation and spread of disease and other dangers caused by harmful chemicals and waste. This not only causes widespread pollution but is harmful for dedicated people who recycle such products. Such waste if mixed with water, leads to leachate formation and leads to toxication of water bodies including drinking water.

**Uses for Recycled Water**

Recycled water is most commonly used for nonpotable (not for drinking) purposes, such as

1. Agriculture
2. Landscape and Golf course irrigation
3. Public parks
4. Cooling water for power plants and oil refineries
5. Processing water for mills, plants
6. Toilet flushing
7. Dust control,
8. Construction activities
9. Concrete mixing
10. Artificial lakes

Although most water recycling projects have been developed to meet nonpotable water demands, a number of projects use recycled water indirectly for potable purposes. These projects include recharging ground water aquifers and augmenting surface water reservoirs with recycled water. In ground water recharge projects, recycled water can be spread or injected into ground water aquifers to augment ground water supplies, and to prevent salt water intrusion in coastal areas.

The use of *grey water* at decentralized sites (see definition) for landscape irrigation and toilet flushing reduces the amount of potable water distributed to these sites, the amount of fertilizer needed, and the amount of wastewater generated, transported, and treated at wastewater treatment facilities. In other words, water reuse saves water, energy, and money. Decentralized water reuse systems are being used more in the arid west where long term drought conditions exist. Successful grey water systems have been operating for many years,. They can meet up to 50% of a property's water needs by supplying water for landscaping. Recycling gray water saves fresh potable water for other uses, reduces the volume of wastewater going to septic systems and wastewater treatment plants, and increases infrastructure capacity for new users.

**Environmental Benefits of Water Recycling**

In addition to providing a dependable, locally-controlled water supply, water recycling provides tremendous environmental benefits. By providing an additional source of water, water recycling can help us find ways to decrease the diversion of water from sensitive ecosystems. Other benefits include decreasing wastewater discharges and reducing and preventing pollution. Recycled water can also be used to create or enhance wetlands.

a) Water Recycling Can Decrease Diversion of Freshwater from Sensitive Ecosystems

Plants, wildlife, and fish depend on sufficient water flows to their habitats to live and reproduce. The lack of adequate flow, as a result of diversion for agricultural, urban, and industrial purposes, can cause deterioration of water quality and ecosystem health. People who reuse water can supplement their demands by using a reliable source of recycled water, which can free considerable amounts of water for the environment and increase flows to vital ecosystems.

b) Water Recycling Decreases Discharge to Sensitive Water Bodies

In some cases, the impetus for water recycling comes not from a water supply need, but from a need to eliminate or decrease wastewater discharge to the ocean, an estuary, or a stream.

- By avoiding the conversion of salt water marsh to brackish marsh, the habitat for two endangered species can be protected.
- Recycled Water May Be Used to Create or Enhance Wetlands and Riparian (Stream) Habitats.
- Water Recycling Can Reduce and Prevent Pollution
- Recycling Water Can Save Energy

**Future of Water Recycling**

Water recycling has proven to be effective and successful in creating a new and reliable water supply without compromising public health. Non-potable reuse is a widely accepted practice that will continue to grow. While water recycling is a sustainable approach and can be cost-effective in the long term, the treatment of wastewater for reuse and the installation of distribution systems at centralized facilities can be initially expensive compared to such water supply alternatives as imported water, ground water, or the use of gray water onsite from homes. Institutional barriers, as well as varying agency priorities and public misperception, can make it difficult to implement water recycling projects. Finally, early in the planning process, agencies must reach out to the public to address any concerns and to keep the public informed and involved in the planning process.

- As water energy demands and environmental needs grow, water recycling will play a greater role in our overall water supply. By working together to overcome obstacles, water recycling, along with water conservation and efficiency can help us to sustainably manage our vital water resources.
- All new single family and duplex residential dwelling units shall include either a separate multiple pipe outlet or a diverter valve, and outside ‗stub-out' installation on clothes washing machine hook-ups, to allow separate discharge of gray water for direct irrigation.
- All new single family residential dwelling units shall include a building drain or drains for lavatories, showers, and bathtubs, segregated from drains for all other plumbing fixtures, and connected a minimum three feet from the limits of the foundation, to allow for future installation of a distributed gray water system.