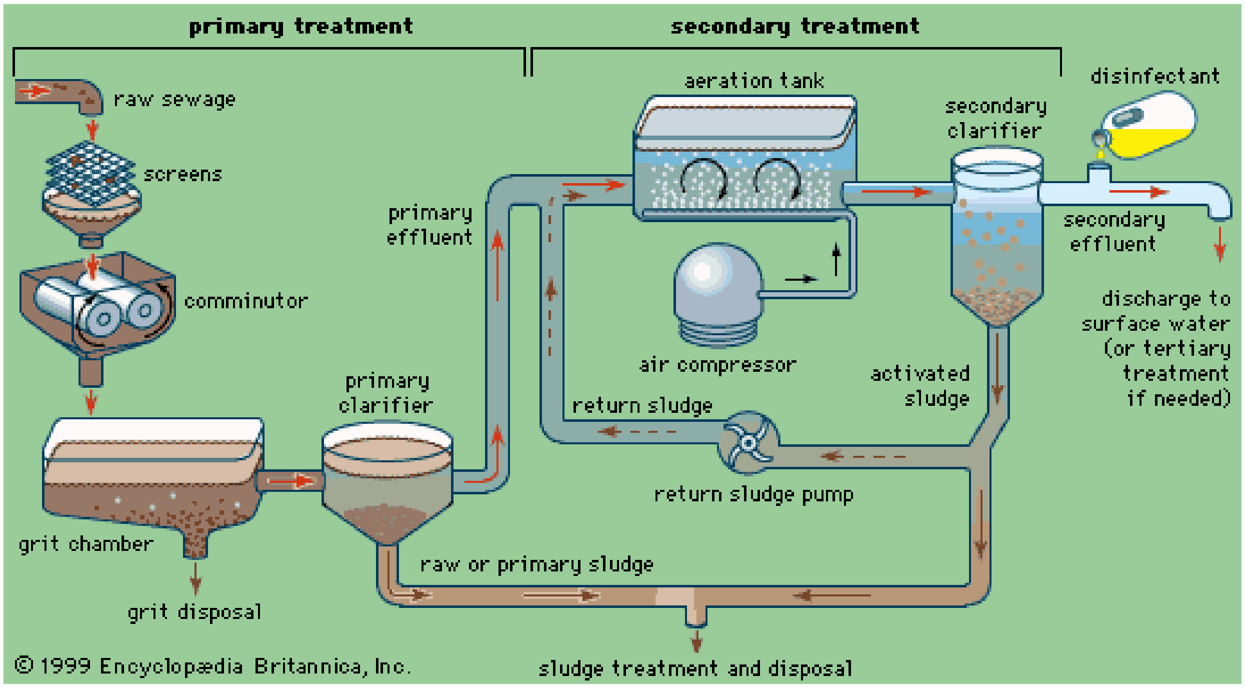
Water Supply and Environmental Engineering Department

Wastewater Treatment Systems Module



Module No: 3192

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1- INTRODUCTION TO WASTEWATER TREATMENT

1.1 General about Wastewater Treatment

Wastewater collected from urban areas and from different industries must ultimately be returned to receiving water bodies or to the land. The complex question of which contaminants in wastewater must be removed is to protect the environment and to what extent must be answered specifically for each case. This requires analyses of local conditions and needs together with the application of scientific knowledge, engineering judgment based on past experience, and consideration of federal, state and local requirements and regulations.

The presence of different pollutants in the wastewater makes it almost impossible to treat all the wastewater in the same manner. Some important contaminants/pollutants of concern in wastewater treatment are given in the Table 1-1.

The contaminants (pollutants) in wastewater are removed by physical, chemical and/or biological means, and the individual methods usually are classified as physical, chemical and biological unit processes or operations.

Treatment methods in which the application of physical forces predominates are known as physical unit operations. Typical physical unit operations are: screening, mixing, flocculation, sedimentation, flotation, and filtration and membrane filter operations.

Treatment methods in which the removal or conversion of contaminants is brought about by the addition of chemicals or by other chemical reactions are known as chemical unit processes.

Neutralization, oxidation, reduction, precipitation, gas transfer, adsorption, ion-exchange, electro-dialysis etc. are the most common examples of these processes used in wastewater treatment.

Treatment methods in which the removal of contaminants is brought about by biological activity are known as biological unit processes.

Biological treatment is used primarily to remove the biodegradable organic substances (colloidal or dissolved) in wastewater. Basically, these substances are converted into gases that can escape to the atmosphere and into biological cell tissue that can be removed by settling. The most common approaches in the biological wastewater treatments are: aerobic processes such as trickling filters, activated sludge, oxidation ponds (or lagoons), and anaerobic processes such as anaerobic lagoons, sludge digestion, etc.

Usually in the municipal wastewater treatment, but also in other wastewater processing all the above mentioned unit operations and processes are grouped together to provide what is known as primary, secondary and tertiary (or advanced) treatment.

The term primary refers to physical unit operations and in some cases to chemical unit processes; secondary refers to biological unit processes; and tertiary refers to combinations of all three.

The contaminants of major interest in wastewater and the unit operations and processes or methods applicable to the removal of these contaminants are shown in Table 1-2.
In addition to the above mentioned classical basic processes, some new directions are also evident in various specific areas of wastewater treatment, including:

(i) modification in treatment operations, processes and concepts
(ii) the changing nature of the wastewater to be treated
(iii) the problem of industrial wastes
(iv) wastewater treatability studies
(v) environmental and energy concerns
(vi) land treatment
(vii) small and individual onsite systems

Table 1-1 Important contaminants of concern in wastewater treatment

Contaminants	Reason for importance
Suspended solids	Suspended solids can lead to the development of sludge deposits and anaerobic conditions when untreated wastewater is discharged in the aquatic environment
Biodegradable organics	Composed principally of proteins, carbohydrates and fats, biodegradable organics are measured most commonly in terms of BOD and COD. If discharged untreated to the environment, their biological stabilization can lead to the depletion of natural oxygen resources and to the development of septic conditions
Pathogens	Communicable diseases can be transmitted by the pathogenic organisms in wastewater
Nutrients	Both nitrogen and phosphorus, along with carbon, are essential nutrients for growth. When discharged to the water these nutrients can lead to the growth of undesirable aquatic life. When discharged in excessive amounts on land they can also lead to the pollution of groundwater
Refractory organics	These organics tend to resist conventional methods of wastewater treatment. Typical examples include surfactants, phenols, and agricultural pesticides
Heavy metals	Heavy metals are usually added to wastewater from commercial and industrial activities and may have to be removed if the wastewater is to be reused
Dissolved inorganic solids	Inorganic constituents such as calcium, sodium, and sulfate are added to the original domestic water supply as a result of water use and may have to be removed if the wastewater is to be reused

Source: Metcalf & Eddy, Wastewater engineering

Table 1-2 Unit operations, processes, and treatment systems

Contaminant	Unit Operation, Unit Process, or Treatment System	Classification
Suspended solids	Screening and comminution	P
	Sedimentation	P
	Flotation	P
	Filtration	P
	Coagulation / sedimentation	C / P
	Land treatment	P
Biodegradable organics	Activated sludge	B
	Trickling filters	B
	Rotating biological contactors	B
	Aerated lagoons	B
	Oxidation ponds	B
	Intermittent sand filtration	P / B
	Land treatment	B / C / P
	Physical / chemical	P / C
Pathogens	Chlorination	C
	Ozonation	C
	Land treatment	P
Nutrients: Nitrogen	Suspended-growth nitrification and denitrification	B
	Fixed-film nitrification and denitrification	B
	Ammonia stripping	C / P
	Ion exchange	C
	Breakpoint chlorination	C
	Land treatment	B / C / P
Phosphorus	Metal salt coagulation/sedimentation	C / P
	Lime coagulation / sedimentation	C / P
	Biological/chemical	C / P
	phosphorus removal	B / C
	Land treatment	C / P
Refractory organics	Carbon adsorption	P
	Tertiary ozonation	C
	Land treatment systems	P
Heavy metals	Chemical precipitation	C

	Ion exchange Land treatment	C C / P
Dissolved inorganic solids	Ion exchange Reverse osmosis Electrodialysis	C P C

*B = biological, C = chemical, P = physical.

The wastewater originating from various sources can be broadly divided into two categories:

1. Biodegradable wastewater

The wastes in general have a predominance of biodegradable organic matter, and are generally treated in a similar manner.

The stabilization of organic matter is accomplished biologically using a variety of microorganisms. The microorganisms are used to convert the colloidal and dissolved carbonaceous organic matter into various gases and into cell tissue. Because cell tissue has a specific gravity slightly greater than that of water, the resulting tissue can be removed from the treated liquid as sludge by gravity settling.

Based on bacterial relationship to oxygen (ability or inability to utilize oxygen as a terminal electron acceptor in oxidation/reduction reactions), the microorganisms can be:

(i) obligate aerobes
(ii) obligate anaerobes
(iii) facultative anaerobes
(iv) denitrifiers

The general term that describes all of the chemical activities performed by a bacterial cell is metabolism which is divided into catabolism and anabolism. Catabolism includes all the biochemical processes by which a substrate (food) is degraded to end products with the release of energy.

Anabolism includes all the biochemical processes by which the bacterium synthesizes new cells.

The type of electron acceptor available for catabolism determines the type of decomposition used by a mixed culture of microorganisms.

Decomposition of wastes and particularly of wastewater can be:

(i) aerobic decomposition
(ii) anaerobic decomposition
(iii) anoxic decomposition

For aerobic decomposition the molecular oxygen (O_2) must be present as the terminal electron acceptor to proceed by aerobic oxidation. The chemical end-products of aerobic decomposition are primarily carbon dioxide (CO_2), water, and new cell material.

Anoxic decomposition occurs when some microorganisms will use nitrate (NO_3^-) as the terminal electron acceptor in the absence of molecular oxygen. Oxidation by this route is called denitrification.

In order to achieve anaerobic decomposition, molecular oxygen and nitrate must not be present as terminal electron acceptors. Sulfate (SO_4^{2-}), carbon dioxide, and organic compounds that can be reduced serve as terminal electron acceptors. The end-products of anaerobic decomposition are hydrogen sulfide (H_2S), mercaptans, methane (CH_4), carbon dioxide, ammonia and water.

2. Non-biodegradable wastewater

The non-biological wastes in general and the wastewater in particular are rich in non-biodegradable matter consisting of solids and liquids in suspended or dissolved form, including various inorganic and organic, many of which may be highly toxic.

Examples are domestic or industrial wastewater containing excessive dissolved solids (minerals), inorganic or organic compounds or naturally occurring organics such as humic and fulvic acids.

Treatment processes are available for removing these contaminants. The physical processes frequently used in engineered systems include sedimentation, filtration and gas-transfer.

Chemical processes include the usage of different chemicals for wastewater treatment. Chemicals may be added to alter equilibrium conditions and cause precipitation of undesirable species. It should be kept in mind that chemical processes are conversion processes and that actual removal is accomplished by physically separating the solid, liquid, or gaseous products of the chemical reactions. The chemical processes frequently used in engineered systems include neutralization, coagulation, flocculation, chemical precipitation and oxidation & reduction.

Some wastewater must be treated by means of highly sophisticated processes and equipment, requiring highly skilled operators, and therefore quite expensive. Such processes are physico-chemical processes and include: demineralization, desalinization, ion-exchange, reverse osmosis, electro-dialysis, adsorption, evaporation, incineration, etc.

1.2 Objectives of Wastewater Treatment

1. To introduce fundamentals of the wastewater treatment plants and their unit operations and processes
2. To provide basic design skills and knowledge on the wastewater treatment plants and their unit operations and processes
3. To experience a design project on a hypothetical wastewater treatment plant

a. Identify kinds and sources of wastewater
b. Describe hazards in wastewater
c. Describe ways of treating wastewater
d. Describe the products of wastewater treatment, including the production and use of biosolids

The overall objectives of wastewater treatment are associated with the removal of pollutants and the protection and preservation of our natural resources.

Specific concern is protection of human health by the destruction of pathogenic organisms present in wastewater prior to treated effluent being discharged to receiving water bodies and land.

1.3 Wastewater Treatment Standards

Effluents from different establishments should be treated before being discharged to receiving bodies so that it should be:

1. Free from materials and heat in quantities, concentrations or combinations which are toxic or harmful to human, animal, aquatic life.
2. Free from anything that will settle in receiving waters forming putrescence or otherwise objectionable sludge deposits, or that will adversely affect aquatic life.
3. Free from floating debris, oil, scum and other materials in amounts sufficient to be noticeable in receiving waters;
4. Free from materials and heat that alone, or in combination with other materials will produce color, turbidity, taste or odour in sufficient concentration to create a nuisance or adversely affect aquatic life in receiving waters;
5. Free from nutrients in concentrations that create nuisance growths of aquatic weeds or algae in the receiving waters.

A significant element in wastewater disposal is the potential environmental impact associated with it.

Environmental standards are developed to ensure that the impacts of treated wastewater discharges into ambient waters are acceptable. Standards play a fundamental role in the determination of the level of wastewater treatment required and in the selection of the discharge location and outfall structures.

Regulations and procedures vary from one country to another and are continuously reviewed and updated to reflect growing concern for the protection of ambient waters. The United States Environmental Protection Agency (USEPA) developed the National Pollutant Discharge Elimination System (NPDES) permit programme in 1972 to control water pollution by regulating point sources that discharge pollutants into waters. Accordingly, industrial, municipal, and other facilities are required to obtain permits if their discharges go directly into surface waters. Under this programme, secondary treatment standards were established by USEPA for publicly owned treatment works, governing the performance of secondary wastewater treatment plants. These technology-based regulations, which apply to all municipal waste-water treatment plants, represent the minimum level of effluent quality attainable by secondary treatment in terms of BOD_5 and TSS removal.

Specific Limits

Effluents discharged to receiving water bodies should achieved the following minimum wastewater quality limits:

Parameter	Effluent Limit
BOD_5	20mg/l
TSS	30mg/l
Nitrates (as Nitrogen)	30mg/l
Phosphate	10mg/l
COD	100mg/l
pH	6 – 9
Faecal coliform	1000MPN/100ml
Residual chlorine	1.5mg/l

MPN- Most Probable Number

1.4 Flow Sheets for Wastewater Treatment Systems

Depending on the contaminants to be removed, an almost limitless number of process combinations can be developed using the unit operations and processes. The term "flow sheet" is used to describe a particular combination of unit operations and processes used to achieve a specific treatment objective. Apart from the analysis of the technical feasibility of the individual treatment methods, the exact flow-sheets configuration will depend on factors such as:

(1) the needs of the client's needs,
(2) the designer's past experience,
(3) regulatory agency policies on the application of specific treatment methods,
(4) the availability of equipment suppliers,
(5) what use can be made of existing facilities,
(6) the availability of qualified operating personnel,
(7) initial construction costs, and
(8) future operation and maintenance costs

Conventional flow sheets for the treatment of wastewater are presented and discussed below.
The choice of a set of treatment methods depends on several factors, including discharge permits and available disposal facilities. For example, where an ocean discharge is used, removal of large debris by screens and of settleable solids by sedimentation may be the only treatment steps that are required. Where treated effluent is to be discharged to an inland stream, complete treatment may be required. Discharges to environmentally sensitive lakes, streams, and estuaries may require additional treatment to remove specific constituents.

Treatment schemes are often identified as primary, secondary, or advanced (also known as tertiary). In *primary treatment*, a portion of the suspended solids and organic matter is removed from the wastewater. This removal is usually accomplished with physical operations such as screening and sedimentation. The effluent from primary treatment will ordinarily contain considerable organic matter and will have a relatively high BOD. The further treatment of the effluent from primary treatment to remove the residual organic matter and suspended material is known as *secondary treatment.* In general, biological processes employing microorganisms are used to accomplish secondary treatment. The effluent from secondary treatment usually has little BOD_5 and suspended solids and may contain several milligrams per liter of dissolved oxygen. When required for water reuse or for the control of eutrophication in receiving waters, *advanced (tertiary) treatment* is used for the removal of suspended and dissolved materials remaining after secondary treatment.

Actually, the distinction between primary, secondary, and advanced treatment is rather arbitrary, as many modern treatment methods incorporate physical, chemical, and biological processes in the same operation. A more rational approach would be to drop these arbitrary distinctions and to focus instead on the optimum combinations of operations and processes that must be used to achieve the required treatment objectives.

Typical flow sheets for the treatment of wastewater are presented in Figure 1-1

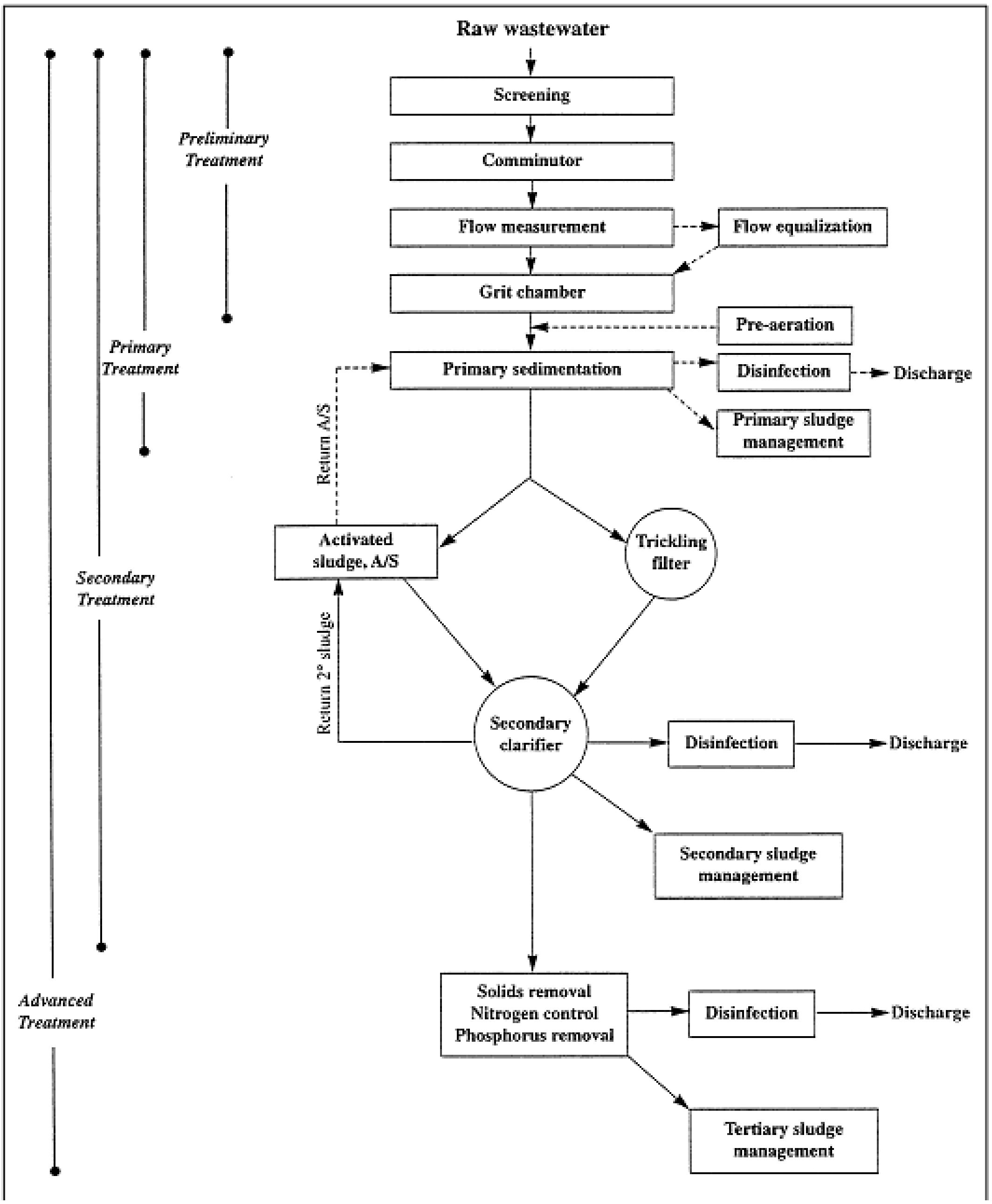


Figure 1-1 Flow chart for wastewater treatment processes

The diagram demonstrates how the treatment plant works and how the different processes are inter-connected to work as one. Each treatment process in the flow sheet will be discussed in chapter 3 and 4.

2- CHARACTERISTICS OF WASTEWATER

Wastewater contains many different substances that can be used to characterize it. The specific substances and amounts or concentrations of each will vary depending on the source. It is difficult to precisely characterize wastewater. Instead, wastewater characterization is usually based on and applied to an average domestic wastewater.

Note: Keep in mind that other sources and types of wastewater can dramatically change the characteristics.

2.1 Physical, Chemical and Bacteriological Characteristic of Wastewater

Wastewater is characterized in terms of its physical, chemical, and biological characteristics.

A. Physical Characteristics

The physical characteristics of wastewater are based on color, odor, temperature, and flow.

1. Turbidity

Sewage is normally turbid, resembling dirty dish water or wastewater from baths having other floating matter like fecal matter, pieces of paper, cigarette-ends, match-sticks, greases, vegetable debris, fruit skins, soaps, etc. The turbidity increases as sewage becomes stronger.

The degree of turbidity can be measured and tested by turbidity rods or by turbid-meters, as is done for testing raw water supplied.

2. Color

Fresh wastewater is usually a light brownish-gray color. However, typical wastewater is gray and has a cloudy appearance. The color of the wastewater will change significantly if allowed to go septic (if travel time in the collection system increases). Typical septic wastewater will have a black color.

3. Odor

Odors in domestic wastewater usually are caused by gases produced by the decomposition of organic matter or by other substances added to the wastewater. Fresh domestic wastewater has a musty odor. If the wastewater is allowed to go septic, this odor will significantly change to a rotten egg odor associated with the production of hydrogen sulfide (H_2S).

4. Temperature

The temperature of wastewater is commonly higher than that of the water supply because of the addition of warm water from households and industrial plants. However, significant amounts of infiltration or storm water flow can cause major temperature fluctuations.

The temperature has an effect on the biological activity of bacteria present in sewage, and it also affects the solubility of gases in sewage. In addition, temperature also affects the viscosity of sewage, which, in turn, affects the sedimentation process in its treatment.

The normal temperature of sewage is generally slightly higher than the temperature of water, because of additional heat added during the utilization of water. The ideal temperature of sewage for the biological activities is 20°c. However, when the temperature is more, the dissolved oxygen content (DO) of sewage gets reduced.

B. Chemical Characteristics

In describing the chemical characteristics of wastewater, the discussion generally includes topics such as organic matter, the measurement of organic matter, inorganic matter, and gases. For the sake of simplicity, chemical characteristics can be described in terms of alkalinity, BOD, chemical oxygen demand (COD), dissolved gases, nitrogen compounds, pH, phosphorus, chloride and solids (organic, inorganic, suspended, and dissolved solids).

1. Total Solids, Suspended Solids and Settleable Solids

Most pollutants found in wastewater can be classified as solids. Wastewater treatment is generally designed to remove solids or to convert solids to a form that is more stable or can be removed. Sewage normally contains very small amount of solids in relation to the huge quantity of water (99.9%). It only contains about 0.05 to 0.1 percent (i.e. 500 to 1000 mg/l) of total solids. Solids present in sewage may be in any of the four forms: suspended solids, dissolved solids, colloidal solids, and settleable solids.

Suspended solids are those solids which remain floating in sewage. Dissolved solids are those which remain dissolved in sewage just as salt in water. Colloidal solids are finely divided solids remaining either in solution or in suspension. Settleable solids are that portion of solid matter which settles out, if sewage is allowed to remain undisturbed for a period of 2 hours. The proportion of these different types of solids is generally found to be as given below:

Inorganic matter consists of minerals and salts, like: sand, gravel, debris, dissolved salts, chlorides, sulphates, etc.

Organic matter consists of:

(i) Carbohydrates such as cellulose, cotton, fiber, starch, sugar, etc.
(ii) Fats and oils received from kitchens, laundries, garages, shops, etc.
(iii) Nitrogenous compounds like proteins and their decomposed products, including wastes from animals, urea, fatty acids, hydrocarbons, etc.

As a general rule, the presence of inorganic solids in sewage is not harmful. They require only mechanical appliances for their removal in the treatment plant. On the other hand, suspended and dissolved organic solids are responsible for creating nuisance, if disposed of, untreated. The amounts of various kinds of solids present in sewage can be determined as follows:

(a) The total amount of solids (S_1 in mg/l) present in a given sewage can be determined by evaporating a known volume of sewage sample, and weighing the dry residue left. The mass of the residue divided by the volume of the sample evaporated will represent the total solids in mg/l, S_1.

(b) The suspended solids (S_2) are those solids which are retained by a filter of 1µm pores; and they are, therefore, also called as filterable solids. Their quantity can be determined by passing a known volume of sewage sample through a glass-fiber filter apparatus, and weighing the dry residue left: The mass of the residue divided by the volume of sample filtered will represent the suspended solids, (S_2), in mg/l.

(c) The difference between the total solids (S_1) and the suspended solids (S_2) will represent nothing but dissolved solids plus colloidal, or non-filterable solids; S_3 where $S_3 = S_1 - S_2$,

(d) Now, the total suspended solids (S_2) may either be volatile or fixed. In order to determine their proportion, the filtered dry residue of step (b) above, is burnt and ignited at about 550°c in an electric muffle furnace for about 15 to 20 minutes. Loss of weight due to ignition will represent the volatile solids in the sample volume filtered through the filter. Let the volatile suspended solids concentration be S_4 (in mg/l).

(e) The difference $S_2 - S_4 = S_5$ will evidently represent the fixed solids.

(f) The quantity of settleable solids (S_6) can be determined easily with the help of a specially designed conical glass vessel called ***Imhoff cone*** (Refer Figure 2-1). The capacity of the cone is 1 liter, and it is graduated up to about 50 ml.

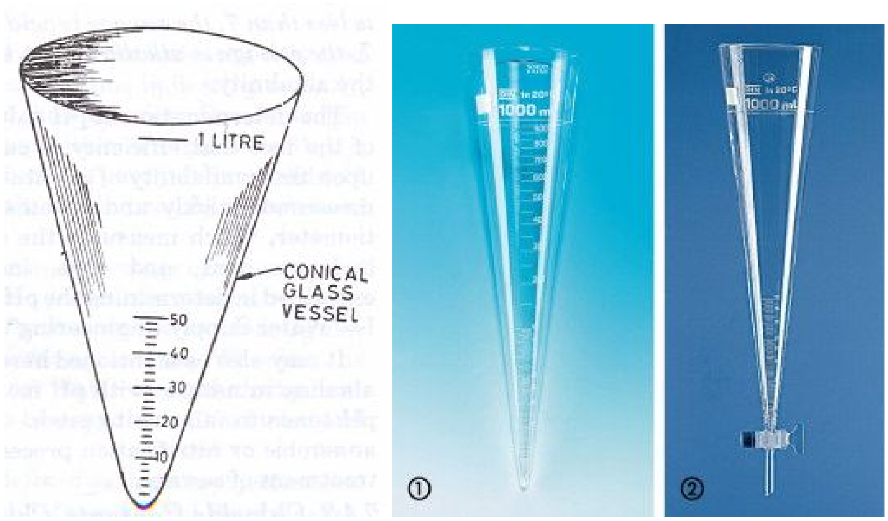


Figure 2-1 Imhoff cone

Sewage is allowed to stand in this Imhoff cone for a period of two hours, and the quantity of solids settled in the bottom of the cone can then be directly read out. However, in order to obtain precise amount of settleable solids, the liquid from the cone should be decanted off, and the settleable solids collected at the bottom of the cone should be dried and weighed. The quantities of different types of solids when determined experimentally as above will help in detecting the strength of sewage, as given in Table 2-1.

2. Alkalinity

This is a measure of the wastewater's capability to neutralize acids. It is measured in terms of bicarbonate, carbonate, and hydroxide alkalinity. Alkalinity is essential to buffer (hold the neutral pH) of the wastewater during the biological treatment processes.

3. pH

This is a method of expressing the acid condition of the wastewater. pH is expressed on a scale of 1 to 14. For proper treatment, wastewater pH should normally be in the range of 6.5 to 9.0.
The determination of pH value of sewage is important, because of the fact that efficiency of certain treatment methods depends upon the availability of a suitable pH value.
It may also be mentioned here that the fresh sewage is generally alkaline in nature (with pH more than 7); but as time passes, its pH tends to fall due to production of acids by bacterial action in anaerobic or nitrification processes. The pH, however, rises upon treatment of sewage.

4. Chloride Contents

Chlorides are generally found present in municipal sewage and are derived from the kitchen wastes, human feces, and urinary discharges, etc. The normal chloride content of domestic sewage is 120mg/l, whereas, the permissible chloride content for water supplies is 250mg/l. However, large amounts of chlorides may enter from industries like ice cream plants, meat salting, etc., thus, increasing the chloride contents of sewage. Hence, when the chloride content of a given sewage is found to be high, it indicates the presence of industrial wastes or infiltration of sea water, thereby indicating the strength of sewage.
The chloride content can be measured by titrating the wastewater (i.e. sewage) with standard silver nitrate solution, using potassium chromate as indicator as is done for testing water supplies.

5. Dissolved gases

These are gases that are dissolved in wastewater. The specific gases and normal concentrations are based upon the composition of the wastewater. Typical domestic wastewater contains oxygen in relatively low concentrations, carbon dioxide, and hydrogen sulfide (if septic conditions exist).

6. Nitrogen compounds

The type and amount of nitrogen present will vary from the raw wastewater to the treated effluent. Nitrogen follows a cycle of oxidation and reduction.

The presence of nitrogen in sewage indicates the presence of organic matter, and may occur in one or more of the following forms:

(a) Free ammonia, called ammonia nitrogen;

(b) Albuminoid nitrogen, called Organic nitrogen;

(c) Nitrites; and

(d) Nitrates

Most of the nitrogen in untreated wastewater will be in the forms of organic nitrogen and ammonia nitrogen.

The free ammonia indicates the very first stage of decomposition of organic matter (thus indicating recently, staled sewage); albuminoid nitrogen indicates quantity of nitrogen present in sewage before the decomposition of organic matter is started; the nitrites indicate the presence of partly decomposed (not fully oxidized) organic matter; and nitrates indicate the presence of fully oxidized organic matter.

The nitrites thus indicate the intermediate stage of conversion of organic matter of sewage into stable forms, thus indicating the progress of treatment. Their presence will show that treatment given to sewage is still incomplete, and the sewage is stale. Whereas, the presence of nitrates indicate the most stable form of nitrogenous matter contained in sewage, thus indicating the well oxidized and treated sewage.

All these different forms of nitrogen, present in sewage, can be tested and measured easily as given below:

The amount of free ammonia present in sewage can be easily measured by simply boiling the sewage, and measuring the ammonia gas which is consequently liberated. The amount of albuminoid nitrogen can be measured by adding strong alkaline solution of potassium permanganate ($KMnO_4$) to the already boiled(removing free ammonia) sewage sample and again boiling the same, when ammonia gas is liberated, which is measured, so as to indicate the amount of albuminoid nitrogen present in sewage. If however an un-boiled sample is used to add $KMnO_4$ before boiling, the evolved ammonia gas will measure the sum total of ammonia nitrogen as well as organic nitrogen; and is known as ***kjedahl nitrogen***.

The amount of nitrites or nitrates present in sewage sample can be measured by colour matching methods. For nitrites, the colour is developed by adding sulphonilic acid and naphthamine; whereas for nitrates, the colour is developed by adding phenol-di-sulphonic acid and potassium hydroxide. The colour developed in waste water is finally compared with the standard colours of known concentrations.

7. Phosphorus

This element is essential to biological activity and must be present in at least minimum quantities or secondary treatment processes will not perform. Excessive amounts can cause stream damage and excessive algal growth. Phosphorus will normally be in the range of 6 to 20mg/L. The removal of phosphate compounds from detergents has had a significant impact on the amounts of phosphorus in wastewater.

8. Presence of Fats, Oils and Greases

Greases, fats and oils are derived in sewage from the discharges of animals and vegetable matter, or from the industries like garages, kitchens of hotels and restaurants, etc.
Such matter form scum on the top of the sedimentation tanks and clog the voids of the filtering media. They thus interfere with the normal treatment methods, and hence need proper detection and removal.
The amount of fats and greases in a sewage sample is determined by making use of the fact that oils and greases are soluble in ether, and when the ether is evaporated, it leaves behind ether-soluble-matter, which represents the quantity of fats and oils. Hence, in order to estimate their amount a sample of sewage is first of all evaporated. The residual solids left are then mixed with ether (hexane). The solution is then poured off and evaporated, leaving behind the fats and greases as a residue which can be easily weighed.

9. Sulphides, Sulphates and Hydrogen Sulphide Gas

The determination of suphides and sulphates in sewage is rarely called far, although their presence reflects aerobic, and/or anaerobic decomposition.
Sulphides and sulphates are formed due to the decomposition of various sulphur containing substances present in sewage, this, decomposition also leads to evolution of hydrogen sulphide gas, causing bad smells and odours, besides causing corrosion of concrete sewer pipes.
In aerobic digestion of sewage, the aerobic and facultative bacteria oxidize the sulphur and its compounds present in sewage to initially form sulphides, which ultimately break down to form sulphate ions (SO_4^{2-}), which is a stable and an unobjectionable end product. The initial decomposition is associated with formation of H_2S gas, which also ultimately gets oxidized to form sulphate ions.
In anaerobic digestion of sewage, however, the anaerobic and facultative bacteria reduce the sulphur and its compounds into sulphides, with evolution of H_2S gas along with methane and

carbon dioxide, thus causing very obnoxious smells and odours. If, however, the quantity of H_2S in raw sewage is below 1ppm, obnoxious odours are not felt.

10. Dissolved Oxygen (DO)

The determination of dissolved oxygen present in sewage is very important, because: while discharging the treated sewage into some river stream, it is necessary to ensure at least 4ppm of DO in it; as otherwise, fish are likely to be killed, creating nuisance near the vicinity of disposal. To ensure this, DO tests are performed during sewage disposal treatment processes.

The DO test performed on sewage before treatment helps in indicating the condition of sewage. It is well known by now that only very fresh sewage contains some dissolved oxygen, which is soon depleted by aerobic decomposition. Also the dissolved oxygen in fresh sewage depends upon temperature. If the temperature of sewage is more, the DO content will be less. The solubility of oxygen in sewage is 95% of that in distilled water.

The DO content of sewage is generally determined by the ***Winkler's method*** which is an oxidation- reduction process carried out chemically to liberate iodine in amount equivalent to the quantity of dissolved oxygen originally present.

11. Bio-Chemical Oxygen Demand (BOD)

Biochemical oxygen demand is used as a measure of the quantity of oxygen required for oxidation of biodegradable organic matter present in the wastewater by aerobic biochemical action. The rate of oxygen consumption in a wastewater is affected by a number of variables: temperature, pH, the presence of certain kinds of microorganisms, and the type of organic and inorganic material in the wastewater. BOD directly affects the amount of DO within the wastewater.

The greater the BOD, the more rapidly oxygen is depleted in the water body, leaving less oxygen available to higher forms of aquatic life. The consequences of high BOD are the same as those for low DO: aquatic organisms become stressed, suffocate, and die.

BOD is one of the most important and useful parameters (measured characteristics) indicating the organic strength of a wastewater. BOD measurement permits an estimate of the waste strength in terms of the amount of dissolved oxygen required to break down the wastewater.

12. Chemical Oxygen Demand (COD)

COD measures the total quantity of oxygen required for oxidation of organics into carbon dioxide and water. The oxygen required to oxidize the organic matter present in a given wastewater can be theoretically computed, if the organics present in wastewater are known. Thus, if the chemical formulas and the concentrations of the chemical compounds present in water are known to us, we can easily calculate the theoretical oxygen demand of each of these compounds by writing the balanced reaction for the compound with oxygen to produce CO_2, H_2O and oxidized inorganic components.

Hence, if the organic compounds and their concentrations are known, the theoretical oxygen demand of the water can be accurately calculated, but it is virtually impossible to know the details of the organic compounds present in any natural raw water or a waste water.

13. Total Organic Carbon

Another important method of expressing organic matter is in terms of its carbon content. Carbon is the primary constituent of organic matter, and hence the chemical formula of every organic compound will reflect the extent of carbon present in that compound. Known concentrations of such chemical compounds in a given wastewater will thus enable us to theoretically calculate the carbon present in that wastewater per liter of solution.

C. Biological Characteristics

The bacterial characteristics of sewage are due to the presence of bacteria and other living micro-organisms, such as algae, fungi, protozoa, etc. The former are more active.

Most of the vast number of bacteria present in sewage (of the order of 5 - 50 billion per liter of sewage) is harmless non-pathogenic bacteria. They are useful and helpful in bringing oxidation and decomposition of sewage. A little number of bacteria, however, is disease producing pathogens, and it is they who constitute the real danger to the health of the public.

In case of sewage, the routine bacteriological tests as performed on water supply samples are generally not performed, because of the high concentration of bacteria present in it. But at the times of epidemiological investigations, certain tests may be useful for separating the pathogenic bacteria. The bacteriological counts may also be useful where the treatment processes are likely to be affected adversely by bactericidal industrial wastewaters.

Table 2-1 Summary of typical domestic wastewater characteristics

Contaminants	Unit	Concentration		
		Weak	Medium	Strong
Total solids (TS)	mg/l	350	720	1200
Total dissolved solids (TDS)	mg/l	250	500	850
Fixed	mg/l	145	300	525
Volatile	mg/l	105	200	325
Suspended solids	mg/l	100	220	350
Fixed	mg/l	20	55	75
Volatile	mg/l	80	165	275
Settleable solids	mg/l	5	10	20
BOD_5, 20^oc	mg/l	110	220	400
TOC	mg/l	80	160	290
COD	mg/l	250	500	1000
Nitrogen (total as N)	mg/l	204	40	85
Organic	mg/l	81	153	35
Free ammonia	mg/l	12	25	50
Nitrites	mg/l	0	0	0
Nitrates	mg/l	0	0	0
Phosphorus (total as P)	mg/l	4	8	15
Organic	mg/l	13	3	5
Inorganic	mg/l	3	5	10
Chlorides	mg/l	30	50	100
Sulfate	mg/l	20	30	50
Alkalinity (as $CaCO_3$)	mg/l	50	100	200
Grease	mg/l	50	100	150
Total coliforms	No/100 ml	10^6 - 10^7	10^7- 10^8	10^7- 10^9
Volatile organic compounds	µg/l	< 100	100 - 400	> 400

Source: Adapted from Metcalf and Eddy Inc., Wastewater Engineering, 3rd edition.

2.2 Measurement of Concentration of Contaminants in Wastewater

Contaminants in wastewaters are usually a complex mixture of organic and inorganic compounds. It is usually impractical, if not nearly impossible to obtain complete chemical analysis of most wastewaters.

However, since it is comparatively easy to measure the amount of oxygen used by the bacteria as they oxidize the wastewater, the concentration of organic matter in the wastewater can easily be expressed in terms of the amount of oxygen required for its oxidation. The most important standard methods for analysis of organic contaminants are:

1. Theoretical Oxygen Demand (ThOD)

This is the theoretical amount of oxygen required to oxidize the organic fraction of the wastewater completely to carbon dioxide and water. The equation for the total oxidation of, say, glucose is:

$$C_6H_{12}O_6 + 6O_2 \longrightarrow 6CO_2 + 6H_2O$$

With C = 12, H = 1 and O = 16, $C_6H_{12}O_6$ is 180 and $6O_2$ is 192; we can thus calculate that the ThOD of, for example, a 300 mg/l solution of glucose is $\frac{192}{180} * 300 = 321\ mg/l$. Because wastewater is so complex in nature its ThOD cannot be calculated, but in practice it is approximated by the chemical oxygen demand.

2. Chemical Oxygen Demand (COD)

The chemical oxygen demand (COD) of a raw water or a wastewater is determined by performing a laboratory test on the given wastewater with a strong oxidant like dichromate solution; and the theoretical computations of COD are only performed on water solutions prepared with the known amounts of specific organic compounds in laboratory situations to compare the theoretical and test results, and to establish the limitations of the test procedures.
The laboratory determination of COD, as said above, lies in using a strong oxidant like potassium dichromate ($K_2Cr_2O_7$) or potassium permanganate ($KMnO_4$) solution to stabilize the organic matter to determine the molecular oxygen used from the oxidant solution in oxidizing the organic matter present in the given wastewater.
In order to perform this test, a known quantity of wastewater is mixed with a known quantity of standard solution of potassium dichromate, and the mixture is heated. The organic matter is oxidized by $K_2Cr_2O_7$ (in the presence of H_2SO_4 (helps to digest/break down the complex molecules)). The resulting solution of $K_2Cr_2O_7$ is titrated with standard ferrous ammonium sulphate [$Fe(NH_4)_2.(SO_4)_2.6H_2O$)], and the oxygen used in oxidizing the wastewater is determined. This is called the chemical oxygen demand (COD) and is a measure of organic matter present in sewage.
The advantage of COD measurements is that they are obtained very quickly (within 3 hours), but they have the disadvantages that they do not give any information on the proportion of the wastewater that can be oxidized by bacteria, nor on the rate at which bio-oxidation occurs.

3. Biochemical Oxygen Demand (BOD)

Oxygen demand of wastewaters is exerted by three classes of materials:

(1) Carbonaceous organic materials usable as a source of food by aerobic organisms

(2) oxidizable nitrogen derived from nitrite, ammonia, and organic nitrogen compounds

which serve as food for specific bacteria (e.g., Nitrosomonas and Nitrobacter).

(3) Chemical reducing compounds, e.g., ferrous ion (Fe^{2+}), sulfites (SO_3^{2-}), and sulfide (S^{2-}) which are oxidized by dissolved oxygen.

For domestic sewage, nearly all oxygen demand is due to carbonaceous organic materials and is determined by BOD dilution test. For effluents subjected to biological treatment, a considerable part of the oxygen demand may be due to nitrification.

BOD Dilution Test

Procedure is given below.

1. Prepare several dilutions of the sample to be analyzed with distilled water of high purity. Recommended dilutions depend on estimated concentration of contaminants responsible for oxygen demand. For highly contaminated waters, dilution ratios (ml of diluted sample/ml of original sample) may be of 100:1. For river waters, the sample may be taken without dilution for low pollution streams, and in other cases dilution ratios of 4:1 may be utilized.
2. Incubation bottles (250ml to 300ml capacity), with ground-glass stoppers are utilized. In the BOD bottle one places:
   (a) the diluted sample (i.e., the “substrate")
   (b) a seed of microorganisms (usually the supernatant liquor from domestic sewage), and
   (c) Nutrient solution for the microorganisms.

   This solution contains sodium and potassium phosphates ammonium chloride (nitrogen and phosphorus are elements needed as nutrients for microorganisms). The pH of the solution in the BOD bottle should be about 7.0 (neutral). Phosphate solution utilized is a buffer. For samples containing caustic alkalinity or acidity, neutralization to about pH 7 is made with dilute H_2SO_4 or NaOH prior to the BOD test.

   For each BOD bottle a control bottle which does not contain the substrate is also prepared.

   Bottles are incubated at 20°c for 5 days. Light must be excluded from the incubator to prevent algal growth that may produce oxygen in the bottle. The DO content before the incubation and after the incubation is thus determined. The difference between concentrations of dissolved oxygen (mg/liter) in control bottle and in sample bottle corresponds to the oxygen utilized in biochemical oxidation of contaminants.

When the dilution water is not seeded:

$$BOD_{5}, mg/lit = \frac{DO_1 - DO_2}{P} \qquad 2.1$$

When the dilution water is seeded:

$$BOD_{5}, mg/lit = \frac{(DO_1 - DO_2) - (DO_{S1} - DO_{S2}) * f}{P} \qquad 2.2$$

Where, DO_1 – dissolved oxygen of the diluted sample immediately after preparation, mg/l

DO_2 – dissolved oxygen of the diluted sample after 5 day incubation at 20°c, mg/l

DO_{S1} – dissolved oxygen of seed control before incubation, mg/l

DO_{S1} – dissolved oxygen of seed control after incubation, mg/l

f – fraction of seeded dilution water volume in sample to volume of seeded dilution water in seed control

P – fraction of wastewater sample volume to total combined volume

As pointed out earlier, this test is conducted to determine the oxygen demand for the first five days on a number of samples, and their average value is taken as the BOD_5 at the test temperature.

If the oxygen supply is made available for periods more than 5 days, it is found that the oxygen is consumed rapidly for 6 or 7 days, and then slows down until the end of about 20 days or more. This value is called ultimate BOD denoted as BOD_u.

Thereafter, it may again accelerate for some time, and again slow down to a very low rate for an indefinite period. The first demand during the first 20 days occurs due to the oxidation of organic matter, and is called *carbonaceous demand* or *first stage demand* or *initial demand*. The latter demand occurs due to biological oxidation of ammonia, and is called *nitrogenous demand* or *second-stage demand*.

A typical BOD curve (BOD vs. incubation time) for oxidation of carbonaceous materials is shown in Figure 2-2 In fact, a sanitary engineer is more concerned with the first stage demand, since the oxygen consumed in its satisfaction is not recoverable. Hence, the term BOD is usually used to mean the first stage BOD, i.e. the demand due to the presence of carbonaceous matter alone.

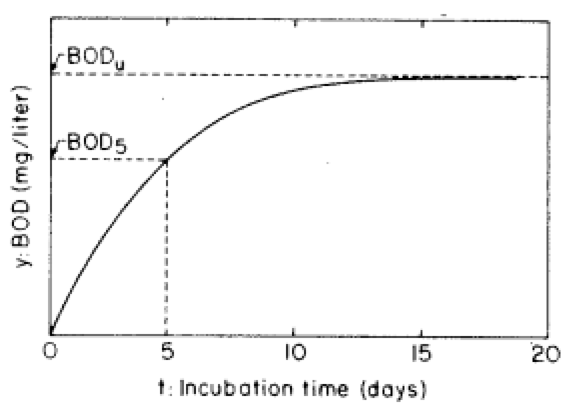


Figure 2-2 Typical BOD curve for oxidation of carbonaceous materials

a. Ratio of COD and BOD

The value of BOD_u is generally lower than that for COD obtained by the standard dichromate oxidation method. The reasons are that:

(1) many organic compounds which are oxidized by $K_2Cr_2O_7$ are not biochemically oxidizable

(2) certain inorganic ions such as sulfide (S^{2-}), thiosulfates ($S_2O_3^{2-}$), sulfites (SO_3^{2-}), nitrites (NO_2^-), and ferrous ion (Fe^{2+}) are oxidized by $K_2Cr_2O_7$, thus accounting for inorganic COD which is not detected by the BOD test.

The $\frac{BOD}{COD}$ ratio will always be less than 1.0; but this value shall approach towards 1.0 with the decreasing amount of non-biological organics.

If this ratio is found to be between 0.92 and 1.0 the wastewater can be considered to be virtually fully biodegradable.

Since, BOD_u is generally not measured and only BOD_5 is measured, the ratio $\frac{BOD_5}{COD}$ usually referred to as $\frac{BOD}{COD}$ ratio becomes more important. Since BOD_5 is about 68% of BOD_u, we can easily state that $\frac{BOD_5}{COD}$ ratio should for fully-biodegradable wastes, vary between 0.92*0.68 = 0.63 and 1.0*0.68 = 0.68. Any wastewater, having its $\frac{BOD}{COD}$ ratio more than 0.63, can hence, be considered to be quite amenable to biological treatment, since it does not contain non-biodegradable organics.

b. Effect of Seeding and Acclimation of Seed on the BOD Test

One of the most frequent reasons for unreliable BOD values is utilization of an insufficient amount of microorganism seed. Another serious problem for industrial wastes is acclimation of seed. For many industrial wastes, the presence of toxic materials interferes with growth of the microorganism population. BOD curves obtained exhibit a time lag period.

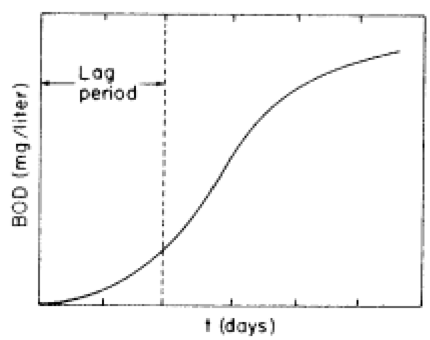


Figure 2-3 Lag period in BOD test

Low BOD values are obtained if adequate corrective action is not taken. It becomes necessary to acclimate the microorganism seed to waste. This is achieved by starting with a sample of settled domestic sewage which contains a large variety of microorganisms, and adding a small amount of industrial effluent. Air is bubbled through this mixture. The operation is performed in bench scale reactors of either continuous or batch type.

The process is repeated with gradual increase in the proportion of industrial waste to domestic sewage, until a microbial culture acclimated to the specific industrial waste is developed. This may be a long and difficult procedure for very toxic industrial wastewaters. When an acclimated culture has been developed, the BOD curve does not present a lag period.

c. Effect of Presence of Algae on the BOD Test

Presence of algae in the wastewater being tested affects the BOD test. If the sample is incubated in the presence of light, low BOD values are obtained owing to production of oxygen by photosynthesis, which satisfies part of the oxygen demand. On the other hand, if incubation is performed in darkness, algae survive for a while. Thus, short-term BOD determinations show the effect of oxygen on them. After a period in the dark, algae die and algal cells contribute to the increase of total organic content of the sample, thus leading to high BOD values. Therefore, the effect of algae on the BOD test is difficult to evaluate.

2.3 Mathematical Model for the BOD Curve

It is desirable to represent the BOD curve by a mathematical model. From kinetic considerations, the mathematical model utilized to portray the rate of oxygen utilization is that of a first-order reaction. Figure 2-4 reveals that the rate of oxygen utilization, given by the tangent to the curve at a given incubation time decreases as concentration of organic matter remaining unoxidized becomes gradually smaller.

The rate at which BOD is satisfied at any time, (i.e. the rate of deoxygenation) depends on temperature and also on the amount and nature of organic matter present in sewage at that time.

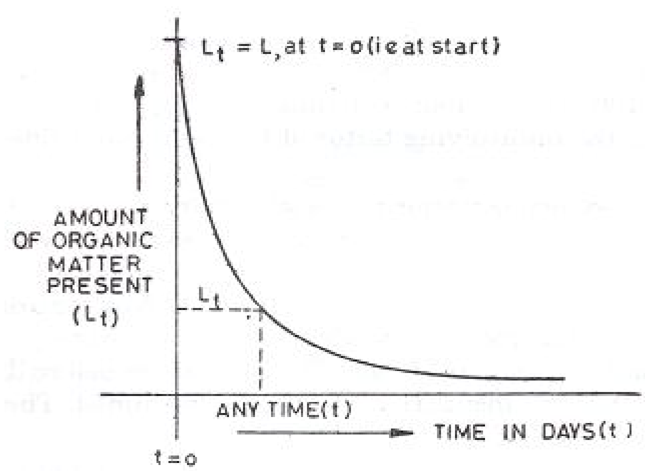


Figure 2-4 1st stage BOD curve

Thus, at a certain temperature, the rate of deoxygenation is assumed to be directly proportional to the amount of organic matter present in sewage at that time (there is proportionality between the rate of oxygen utilization and that of destruction of organic matter by biological oxidation); i.e.

$$\frac{dL_t}{dt} = -KL_t \qquad 2.3$$

Where, L_t = oxygen equivalent of carbonaceous oxidizable organic matter present in sewage after t days from the start of oxidation, in mg/l.

$\frac{dL_t}{dt}$ = rate of disappearance of organic matter by aerobic biological oxidation

t = time in days.

K = rate constant signifying the rate of oxidation of organic matter, and it depends upon the nature of organic matter and temperature. Its unit is per day.

Minus sign indicates that with the passage of time (i.e., increase in t) the value of L_t decreases.

Integrating the above equation, we get

$$\int \frac{dL_t}{L_t} = \int -Kdt$$

$$\log_e L_t = -Kt + C \qquad 2.4$$

Where, C is a constant of integration, and can be evaluated from the boundary conditions at the start i.e.

When t = zero (0), i.e. at start L_t = L (say). Substituting in the above equation, we have

$$\log_e L_t = -K * 0 + C$$

$$C = \log_e L$$

Substituting this value of C in equation 2.4,

$$\log_e L_t = -Kt + \log_e L$$

$$\log_e L_t - \log_e L = -Kt$$

$$\log_e \frac{L_t}{L} = 2.3 * \log_{10} \frac{L_t}{L} = -Kt$$

$$\log_{10} \frac{L_t}{L} = \frac{-Kt}{2.3} = -0.434Kt$$

Using 0.434K = K_D

Where, K_D is the De-oxygenation constant or the BOD rate constant (on base 10) at the given temperature = 0.434K.

We have

$$\log_{10} \frac{L_t}{L} = -K_D t$$

$$\frac{L_t}{L} = 10^{-K_D t} \qquad 2.5$$

Now, L is the organic matter present at the start of BOD reaction, (expressed as oxygen equivalent) and L_t is the organic matter left after t days; which means that during t days, the quantity of organic matter oxidized = L - L_t.

If Y_t represents the total amount of organic matter oxidized in t days (i.e. the BOD of t days), then we have

$$Y_t = L - L_t = L\left(1 - \frac{L_t}{L}\right)$$

$$\frac{Y_t}{L} = 1 - \frac{L_t}{L}$$

$$\frac{L_t}{L} = 1 - \frac{Y_t}{L}$$

Substituting this value to the above equation, we get

$$\frac{L_t}{L} = 1 - \frac{Y_t}{L} = 10^{-K_D t}$$

$$\frac{Y_t}{L} = 1 - 10^{-K_D t}$$

$$Y_t = L * (1 - 10^{-K_D t}) \qquad 2.6$$

This is an important equation. Y_t is the oxygen absorbed in t days, i.e. BOD of t days.

The ultimate first stage BOD (Y_u) would be obtained from the above equation, when we substitute t = ∞ days in it.

$$Y_u = L * (1 - 10^{-K_D \infty})$$

$$Y_u = L * \left(1 - \frac{1}{10^{\infty}}\right) = L \qquad 2.7$$

Hence, the ultimate first state BOD (Y_u) of a given sewage is equa1 to the initial oxygen equivalent of the organic matter present in this sewage (L). This is a fixed quantity, and does not depend upon the temperature of oxidation.

The value of K_D however, determines the speed of the BOD reaction, without influencing the ultimate BOD, as shown in Figure 2-5:

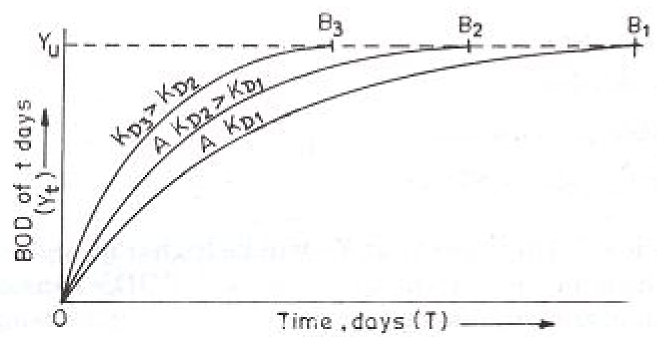


Figure 2-5 BOD exertion as a function of K_D

The figure shows that the coefficient of deoxygenation is different at different temperatures, but finally, Y_u is constant.

It is found to vary with temperature of sewage, and this relationship is approximately given by the equation

$$K_{D(T)} = K_{D(20)} * 1.047^{T-20} \quad 2.8$$

Where, $K_{D(20°)}$ = Deoxygenation constant at 20°c. Its numerical value varies between (0.05 to 0.2) per day depending upon the nature of the organic matter present in sewage. Simple compounds such as sugars and starches are easily utilized by the micro-organisms, and have a high K_D rate, while complex molecules such as phenols are difficult to assimilate and hence have low K_D values. Some typical K_D values are given in Table 2-2.

$K_{D(T)}$ = Deoxygenation constant at temperature T°c.

Table 2-2 Typical values of K_D at 20°c for various types of waters and wastewaters

Water type	K_D value per day
Tap waters	< 0.05
Surface waters	0.05 - 0.1
Municipal wastewaters	0.1 - 0.15
Treated sewage effluents	0.05 - 0.1

Equation 2.8 shows that K_D will be higher at higher temperatures, which means that the speed at which BOD is consumed in the oxidation of the organic matter, is higher at higher temperatures. This means that the entire carbonaceous organic matter will get oxidized quickly and in lesser time at higher temperatures.

Equation 2.6 is called the first stage equation of BOD reaction, and is represented graphically by the curve OAB of Figure 2-6.

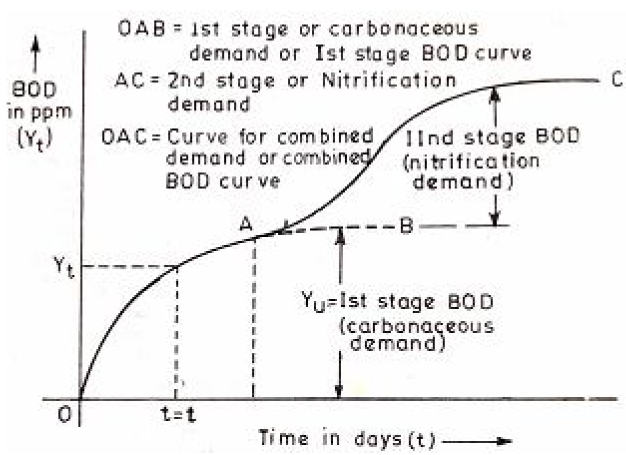


Figure 2-6 Combined BOD curve

By comparing Figure 2-4 and Figure 2-6, it can be seen that the curve of Figure 2-6 is nothing but the reciprocal of curve in Figure 2-4. This is because the oxygen used in satisfying BOD is in direct ratio of the amount of organic matter oxidized, which means reciprocal of the curve of Figure 2-4.

The portion AC of the curve of Figure 2-6 represents the nitrification stage, which follows the carbonaceous stage, so that the BOD curve for the complete oxidation is represented by OAC.

Note: The equations given above are only for the important first stage BOD, and have nothing to do with the second stage BOD, with which we are not concerned here.

Laboratory estimation of K_D value

The BOD rate constant K_D used in eqn. (2.6) can be computed from BOD values measured at various times. The sewage samples are tested for BOD at different times (t), such as after 0.5, 1.0, 1.5, 2.0, 3.0, 5.0 days. A graph is now plotted between the values of time t in days on X-axis, and the values of function $\sqrt[3]{\frac{\text{t(in days)}}{\text{BOD(in mg/l)}}}$ on Y-axis. The best fit line drawn through these points is used to calculate the K_D rate by the following relationship:

$$K_D = 2.61 \frac{A}{B} \qquad 2.9$$

Where, K_D = rate constant per day

A = slope of the line.

B = intercept of the line on Y-axis.

Example 2-1

Compute

(a) the theoretical oxygen demand; and

(b) the organic carbon concentration of a water that contains the following chemical compounds:

  (i) glucose ($C_6H_{12}O_6$) = 200 mg/l ; and

  (ii) benzene ($C_6H_{6)}$ = 25 mg/l

(c) What is the formula weight of the organic matter in this solution?

Example 2-2

The BODs of a wastewater is 150mg/l at 20°c. The k value is known to be 0.23 per day. What would BOD_8 be, if the test was run at 15°?

Example 2-3

The 5 day 30°c BOD of sewage sample is 1l0mg/l. Calculate its 5 days 20°c BOD. Assume the deoxygenation constant at 20°c K_{20} as 0.1.

Example 2-4

Calculate 1 day 37°c BOD of sewage sample whose 5 day 20°c BOD is 100 mg/l. Assume K_D at 20°c as 0.1.

Example 2-5

The BOD_5 of a waste has been measured as 600mg/l. If K = 0.23/day, what is the ultimate BOD_u of the waste. What proportion of the BOD_u would remain unoxidised after 20 days.

Example 2-6

The following observations were made on a 3% dilution of waste water.

Dissolved oxygen (DO) of aerated water used for dilution = 3.0 mg/l

Dissolved oxygen (DO) of diluted sample after 5 days incubation = 0.8 mg/l

Dissolved oxygen (DO) of original sample = 0.6 mg/l.

Calculate the BOD of 5 days and ultimate BOD of the sample assuming that the deoxygenation coefficient at test temp. is 0.1.

3- PRELIMINARY AND PRIMARY WASTEWATER TREATMENT METHODS

3.1 Preliminary Treatment

Preliminary treatment consists solely in separating the floating materials (like dead animals, tree branches, papers, pieces of rags, wood, etc.), and also the heavy settleable inorganic solids. It also helps in removing the oils and greases, etc. from the sewage. This treatment reduces the BOD of the wastewater, by about 15 to 30%. The processes used are:

- Screening for removing floating papers, rags, clothes, etc
- Grit chambers or Detritus tanks for removing grit and sand; and
- Skimming tanks for removing oils and greases

3.1.1 Screening

Screening is the very first operation carried out at a sewage treatment plant, and consists of passing the sewage through different types of screens, so as to trap and remove the floating matter, such as pieces of cloth, paper, wood, cork, hair, fiber, kitchen refuse, fecal solids, etc. present in sewage. These floating materials, if not removed, will choke the pipes, or adversely affect the working of the sewage pumps. Thus, the main idea of providing screens is to protect the pumps and other equipments from the possible damages due to the floating matter of the sewage.

Screens should preferably be placed before the grit chambers (described in the next article). However, if the quality of 'grit' is not of much importance, as in the case of land fillings, etc., screens may even be placed after the grit chambers. They may sometimes be accommodated in the body of the grit chambers themselves.

1. Types of Screens, their Designs and Cleaning

Depending upon the size of the openings, screens may be classified as coarse screens, medium screens, and fine screens.

(i) **Coarse screens** are also known as Racks, and the spacing between the bars (i.e. opening size) is about 50 mm or more. These screens do help in removing large floating objects from sewage. They will collect about 6 liters of solids per million liter of sewage. The material separated by coarse screens, usually consists of rags, wood, paper, etc., which will not putrefy, and may be disposed of by incineration, burial, or dumping.

(ii) **In medium screens**, the spacing between bars is about 6 to 40 mm. These screens will ordinarily collect 30 to 90 liters of material per million liter of sewage. The screenings usually contain some quantity of organic material, which may putrefy and become offensive, and must, therefore, be disposed of by incineration, or burial (not by dumping).

Rectangular shaped coarse and medium screens are now-a-days widely used at sewage treatment plants. They are made of steel bars, fixed parallel to one another at desired spacing on a rectangular steel frame, and are called ***bar screens***. The screens are set in a masonry or R.C.C. chamber, called the screen chamber.

Now-a-days, these screens are generally kept inclined at about 30 to 60° to the direction of flow, so as to increase the opening area and to reduce the flow velocity; and thus making the screening more effective. While designing the screens, clear openings should have sufficient total area, so that the velocity through them is not more than 0.8 to 1m/sec. This limit placed on velocity limits the head loss through the screens, and, thus, reduces the opportunity for screenings to be pushed through the screens.

The material collected on bar screens can be removed either manually or mechanically. Manual cleaning is practiced at small plants with hand operated rakes. The inclined screens help in their cleaning by the upward stroke of the rake. Large plants, however, use mechanically operated rakes, which move over the screens, either continuously or intermittently.

The cleaning of screens by rakes will be hindered by cross bars, if at all provided. They are, therefore, generally avoided.

Screens are sometimes classified as fixed or movable, depending upon whether the screens are stationary or capable of motion.

Fixed screens are permanently set in position. A most commonly used bar type screen is shown in Figure 3-1.

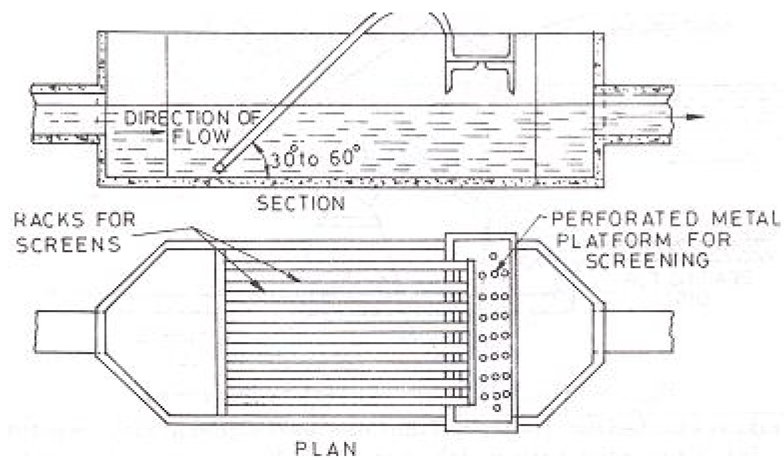


Figure 3-1 Fixed bar type coarse or medium screen

Movable screens are stationary during their operating periods. But they can be lifted up bodily and removed from their positions for the purpose of cleaning. A common movable bar medium screen is a 3-sided cage with a bottom of perforated plates. It is mainly used in deep pits ahead of pumps.

(iii) Fine Screens have perforations of 1.5 mm to 3 mm in size.

The installation of these screens proves very effective, and they remove as much as 20% of the suspended solids from sewage. These screens, however, get clogged very often, and need frequent cleaning. They are, therefore, used only for treating the industrial wastewaters, or for treating those municipal wastewaters, which are associated with heavy amounts of industrial wastewaters. These screens will considerably reduce the load on further treatment units.

Brass or Bronze plates or wire meshes are generally used for constructing fine screens. The metal used should be resistant to rust and corrosion

The fine screens may be disc or drum type, and are operated continuously by electric motors.

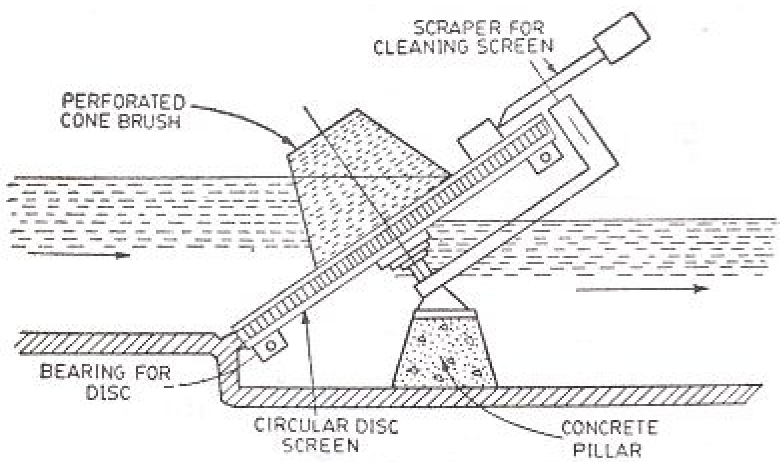


Figure 3-2 Reinsch-Wurl screen (disc type fine screen)

Example 3-1

Estimate the screen requirement for a plant treating a peak flow of 6.0 million liters per day of sewage.

Solution

$$\text{Peak flow} = 60\ \text{Ml/day}$$

$$= \frac{60 * 106}{1000}\text{cu.m/day} = 0.694\ \text{m}^3/\text{sec}$$

Assuming that the velocity through the screens (at peak flow) is not allowed to exceed 0.8m/sec,

The net area of screen openings required

$$= \frac{0.694}{0.8} = 0.87\text{m}^2$$

Using rectangular steel bars in the screen, having 1cm width, and placed at 5cm clear spacing,

The gross area of the screen required:

$$= \frac{0.87 * 6}{5} = 1.04 cm^2$$

Assuming that the screen bars are placed at 60° to the horizontal,

The gross area of the screen needed

$$= \frac{1.04}{\frac{\sqrt{3}}{2}} = 1.2 \text{ m}^2$$

Hence, a coarse screen of 1.2 m^2 area is required.

While designing the screen, we have also to design its cleaning frequency. The ***cleaning frequency*** is governed by the head loss through the screen. The more the screen openings are clogged, more will be the head loss through the screen. Generally, not more than half the screen clogging is allowed. To know whether the screen has been clogged and needs cleaning, we can check or measure the head loss.

The head loss through the cleaned screen and half-cleaned screen can be computed as follows:

Velocity through the screen = 0.8m/sec.

Velocity above the screen

$$= \frac{0.8 * 5}{6} = 0.67 \text{ m/sec}$$

Head loss through the screen

$$= 0.0729 * (V^2 - v^2) \qquad (3.1)$$

$$= 0.0729 * (0.8^2 - 0.67^2) = 0.0134\text{m}$$

When the screen openings get half clogged, then

The velocity through the screen

$$v = 0.8 * 2 = 1.6\text{m/sec}$$

$$\text{Head loss} = 0.0729 * (1.6^2 - 0.67^2) = 0.1538 \cong 0.15\text{m}$$

This shows that when the screens are totally clean, the head loss is negligible i.e. about 1.3cm only; whereas, the head loss shoots up to about 15cm at half the clogging. The screens should therefore be cleaned frequently, as to keep the head loss within the allowable range.

2. Comminutors

Comminutors or Shredders are the patented devices, which break the larger sewage solids to about 6mm in size, when the sewage is screened through them. Such a device consists of a revolving slotted drum, through which the sewage is screened (Figure 3-3). Cutters mounted on the drum, shear the collected screenings against a comb, until they are small enough to pass through 5 mm to 10 mm wide slots of the drum. These are usually arranged in pairs to facilitate repairs and maintenance. Comminutors are of recent origin, and eliminate the problem of

disposal of screenings, by reducing the solids to a size which can be processed elsewhere in the plant. They should always be preceded by grit chambers to prevent their excessive wear.
Such devises are used only in developed countries like, and generally not adopted in our country.

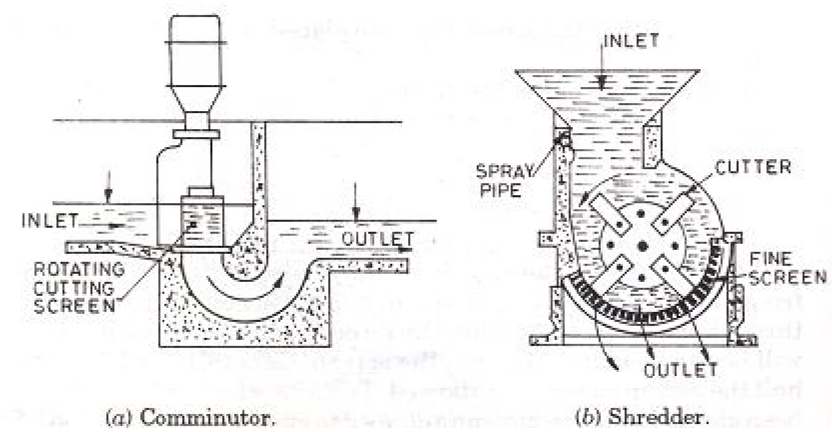


Figure 3-3 Comminutor or shredder

3. Disposal of Screenings

The material separated by screens is called the screenings. It contains 85 to 90% of moisture and other floating matter. It may also contain some organic load which may putrefy, causing bad smells and nuisance. To avoid such possibilities, the screenings are disposed of either by burning, or by burial, or by dumping. The dumping is avoided when screenings are from medium and fine screens, and are likely to contain organic load, as pointed out earlier. The screenings may also sometimes be broken up by a Comminutor and then passed on to the grit chamber.
Burning of the screenings is done in the incinerators, similar to those used for burning garbage. The process of burning is called **Incineration**. The screenings are first dried with sun's heat by spreading on ground or by compressing through hydraulic or other presses, so as to reduce the moisture content to about 60%. The incineration is carried out at temperatures of about 760 to 815°c. This will avoid bad smells.
The screenings may also be disposed of by burial. The process is technically called **composting**. In this process, the screenings are buried in 1 to 1.5m deep trenches, and then covered with 0.3 to 0.45m of porous earth. In due course of time, oxidation-reduction of screenings will take place, and the contents can be used as manure.
Another method of disposing of the screenings is by **dumping** them in low lying areas (away from the residential areas) or in large bodies of water, such as sea. Dumping in sea will be suitable only where strong forward currents do exist to take the dumped material away from the shore line. The dumping on land for raising low lying areas is also adopted only when screenings are from the course screens and not from the medium or fine screens, and as such not containing much organic load.

Digestion of screenings along with the sewage sludge in a sludge digestion tank has also been tried, but not found successful.

3.1.2 Grit Removal Basins

Grit removal basins, such as Grit chambers or Grit channels or Detritus tanks are the sedimentation basins placed in front of the wastewater treatment plant. The grit chamber remove the inorganic particles (specific gravity about 2.65 and nominal diameter of 0.15 to 0.20mm or larger) such as sand, gravel, grit, egg shells, bones, and other non-putresible materials that may clog channels or damage pumps due to abrasion, and to prevent their accumulation in sludge digesters.

Grit chambers are, in fact, nothing but like sedimentation tanks, designed to separate the intended heavier inorganic materials by the process of sedimentation due to gravitational forces, and to pass forward the lighter organic material. (The organic material is not allowed to settle in this process, as otherwise, the organic matter gets entangled with the inorganic matter, causing septicity of sewage and requiring unnecessary labor and expenses for disposal of removal.)

Actually, grit will also include smaller mineral particles that may settle, as well as non-putrescible organic matter, such as rags, coffee grounds, vegetable cuttings, ash clinker, wood pieces, and tea leaves. Even though, some of the grit components, such as coffee grounds are organic, they are essentially non-biodegradable over time spans involved here in grit collection and disposal. The quality and quantity of grit in the sewage determine the design factors and choice of grit removal method.

The amount of grit collected is a function of the removal device, its operation, and the quantity of grit in the sewage, and therefore, it varies over a wider range. The grit quantity may vary between 0.004 - 0.037m^3/1000m^3 of sewage for separate sewage system; while this may range between 0.004 - 0.180 m^3/1000 m^3 for combined sewage system.

Generally, grit chambers are designed to remove all particles with a nominal diameter of 0.02mm having settling velocity of about 2.3cm/s (at 10ºc); although some grit removal devices are designed to remove 0.15mm sand particles having settling velocity of about 1.3cm/s (at 10°c).

It is not at all desirable to remove any organic matter in grit chambers, because no further treatment of removed grit is provided. The grit chamber must hence, be designed to scour the lighter organic particles, and while the heavier grit particles remain settled.

Grit chambers or Grit channels, as they are sometimes called, are designed to have constant velocity horizontal flow at varying discharges. The constant velocity is achieved by providing a velocity control section, such as a proportional flow weir at the effluent end of a rectangular chamber; or a parshall flume (venturi flume) in a parabolic (or V) shaped chamber, as discussed below:

i. Constant Velocity Horizontal Flow Grit Chambers

Such a grit channel is an enlarged channel or a long basin, in which the cross-section is increased, so as to reduce the flow velocity of sewage to such an extent that the heavy inorganic materials do settle down by gravity, and the lighter organic materials remain in suspension, and, thus, go out along with the effluent of the grit basin. The important point in the design of the grit basins is that the flow velocity should neither be too low as to cause the settling of lighter organic matter, nor should it be so high as not to cause the settlement of the entire silt and grit present in sewage. The flow velocity should also be enough to scour out the settled organic matter, and reintroduce it into the flow stream. Such a critical scouring velocity is, infact, given by the modified Shield's formula, which states that

Critical scour velocity

$$= V_H = 3 \text{ to } 4.5\sqrt{gd(G-1)} \qquad (3.2)$$

For grit particles of 0.2mm (d), the above formula gives critical velocity values of 0.11 to 0.25m/sec. This fixes the limits for optimum flow velocity for design of grit basins. In practice, a flow velocity of about 0.25 to 0.3m/sec is adopted for the design of grit basins.

In order to prevent large increase in flow velocity at peak hours due to increased discharge, and thus, to avoid the scouring of the settled grit particles from the bottom, it is preferable to design the grit chambers for DWF (Dry weather flow), and to provide additional units for taking increased discharge at peak hours. If, however, a single unit is to be designed, or there are large variations in discharge, then the grit chamber is designed for generating optimum velocity at peak discharge and a velocity control section, such as a proportional flow weir or a parshall flume venturi flume), is provided at the lower (effluent) end of the grit channel, which helps in varying the flow area of the section in direct proportion to the flow, and thus, helps to maintain a constant velocity in the channel (within the permissible limits of ±5 to 10% over the designed value), even at varying discharges.

When a proportioning flow weir is used as a velocity control device, then a rectangular cross-section is required for the grit channel; but however, when a parshall flume is used as a velocity control device, then a parabolic cross-section is required for the grit channel, in order to keep the flow velocity constant, as shown in Figure 3-4(a) and (b).

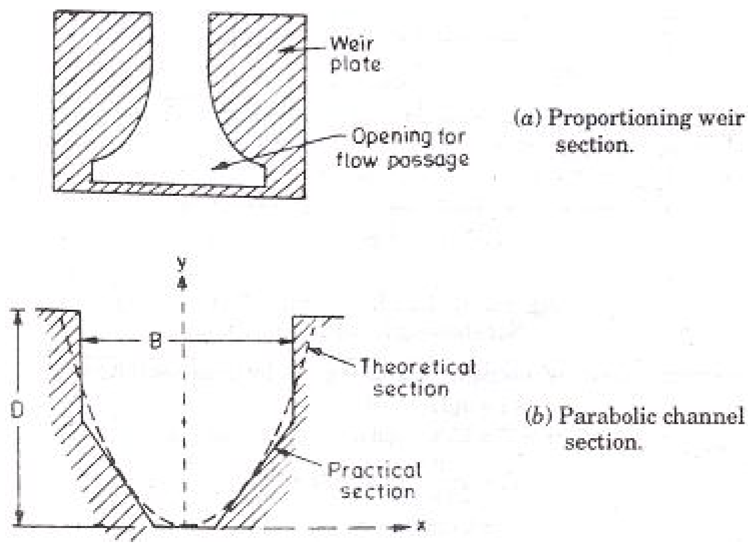


Figure 3-4 Velocity control sections for horizontal grit channels

1. **Design of a Rectangular Grit Chamber provided with a Proportioning Weir at Effluent End**

The depth and detention time provided for a grit basin are inter dependent, and are based on the considerations of settling velocity of inorganic particles through water. A detention time of about 40 to 60 seconds is generally sufficient for a water depth of about 1 to 1.8m. After fixing the depth and the detention time, we can easily design the dimensions of a rectangular chamber, as its length will then be equal to velocity * detention time.

As stated earlier, generally two to three separate chambers in parallel (as shown in Figure 3-5) should be provided; one to pass the low flow, and the other to pass (along with the first of course) the high flow. This will also help in manual cleaning of the chambers, as one unit can work, while the other is shut down for cleaning.

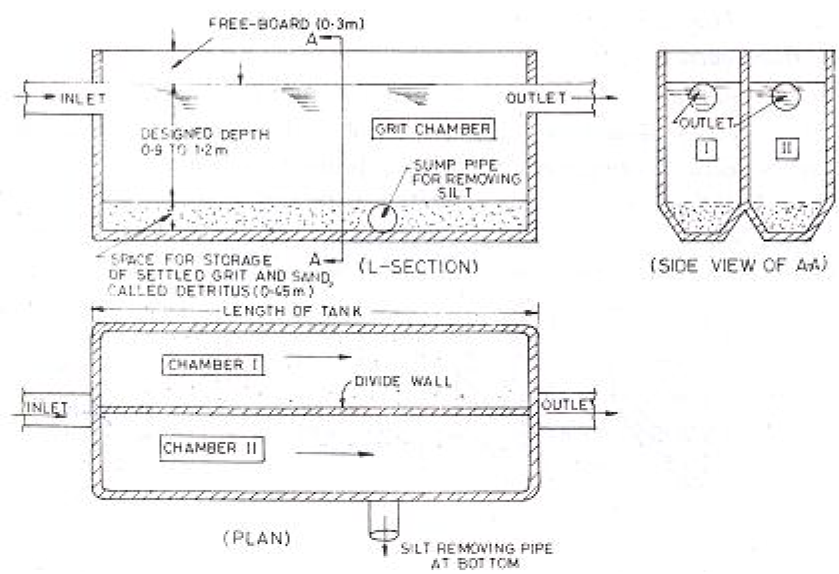


Figure 3-5 Modem rectangular grit chamber (not showing scraper arrangements)

The grit chambers can be cleaned periodically at about 3 weeks interval, either manually, mechanically or hydraulically hand cleaning is done only in case of smaller plants (of capacity less than about 4.5 million liters per day), while mechanical or hydraulic cleaning is adopted for larger plants. In manual cleaning, grit is removed by shovels, etc., by hand; in mechanical cleaning, grit is removed with the help of machines; and in hydraulic cleaning, grit is removed by the force of water jet directed from a central point and removed through the pipes in the side walls or bottom of the chamber.

The removed grit may contain some organic matter, and can be washed prior to its disposal, if necessary, by using certain patented machines, and the wash water returned to the plant influent. Washed grit may still contain about 1 to 5% of putrescible organic matter.

The silt and grit, etc. removed by the grit chambers can be easily disposed of either by burial or burning (incineration) or for raising law lying areas by dumping. It cannot be used for preparation of concrete, as it contains sufficient organic matter.

Example 3-2

A grit chamber is designed to remove particles with a diameter of 0.2mm, specific gravity 2.65. Settling velocity for these particles has been found to range from 0.016 to 0.022m/sec, depending on their shape factor. A flow through velocity of 0.3m/sec will be maintained by proportioning weir. Determine the channel dimensions for a maximum wastewater flow of 10,000cu m/day.

Solution

Let us provide a rectangular channel section, since a proportional flow weir is provided for controlling velocity of flow.

Horizontal velocity of flow = V_h = 0.3m/sec.

Settling velocity is between 0.016 to 0.022 m/sec, and hence let it be 0.02m/sec.

$$Q = \text{velocity} * \text{crosssection} = V_h * A$$

Where, Q = 10,000cu m/day = 0.116m3/sec

Therefore,

$$0.116 = 0.3 * A$$

$$A = \frac{0.116}{0.3} = 0.385\ m^2$$

Assuming a depth of 1m, we have the width (B) of the basin as

$$1 * B = 0.385$$

$$B = 0.385m \cong 0.4m$$

Settling velocity

Vs = 0.02m/sec

$$\text{Detention time} = \frac{\text{Depth of the basin}}{\text{Settling velocity}} = \frac{1}{0.02} = 50\text{sec}$$

$$\text{Length of the tank} = V_h * \text{Detention time} = 0.3m/s * 50s = 15m$$

Hence, use a rectangular tank, with dimensions:

Length (L = 15m) Width (B = 0.4m) and Depth (D = 1.0m)

Example 3-3

Design a suitable grit chamber cum Detritus tank for a sewage treatment plant getting a dry weather flow from a separate sewerage system @4001/s. Assume the flow velocity through the tank as 0.2m/sec and detention period of 2 minutes. The maximum flow may be assumed to be three times of dry weather flow.

Solution

The length of the tank

$$= \text{Velocity} * \text{Detention time} = 0.2 * (2 * 60) = 24m$$

Assuming that each detritus tank is designed for passing Dry Weather Flow (DWF),

The discharge passing through each tank

$$= 400l/s = 0.4m^3/sec$$

Therefore, Cross-sectional area required

$$A = \frac{\text{Discharge}}{\text{Velocity}} = \frac{0.4}{0.2} = 2m^2$$

Assuming the water depth in the tank to be 1.2m,

The width of the tank

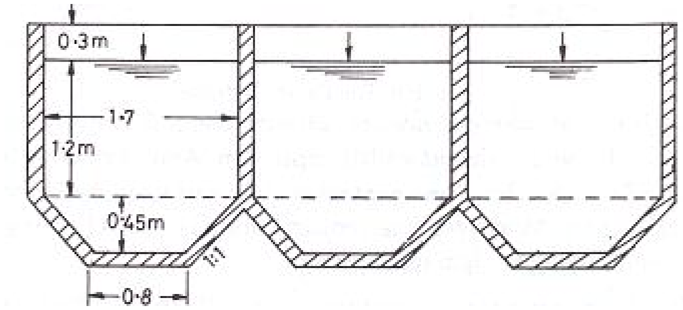
$$= \frac{\text{Area of cross section}}{\text{Depth}} = \frac{2}{1.2} = 1.67m \cong 1.7m$$

Hence, use a Detritus tank with ***24m*1.7m*1.2m*** size.

At the top, a free-board of 0.3m may be provided; and at the bottom, a dead space depth of 0.45m for collection of detritus may be provided.

Thus, the overall depth of the tank = 1.2 + 0.3 + 0.45 = 1.95m.

The tank will be 1.7m wide up to 1.5m depth, and then the sides will slope down to form an elongated trough of 24m length and 0.8m width at the bottom with rounded corners, as shown in figure below.



2. Design of Parabolic Grit Chamber provided with a Parshall Flume

i. Parshall Flume

A parshall flume, also called a venturi-flume, is a horizontally constricted vertical throat in an open channel, as shown in Figure 3-6. Such a venturi-flume, as we know, can be used as a discharge measuring device, and also as a velocity control device. This device is made use of for its latter purpose in a grit channel. .

The venturi-flume, as a velocity control device, is preferable (to the proportional flow weir, etc., as it involves negligible head loss, and can also work under submerged conditions for certain limits.

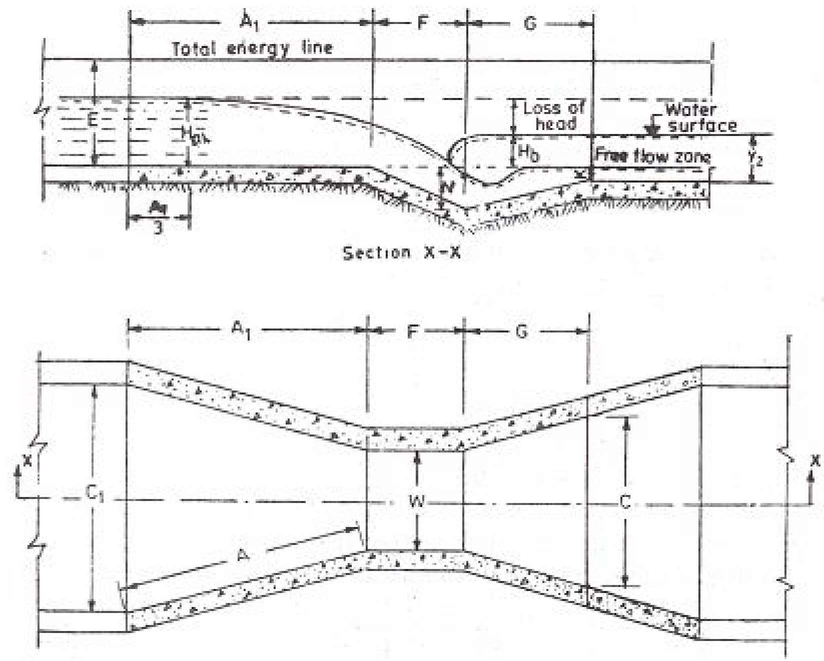


Figure 3-6 Parshall flume

These *limits of submergence* are: 50% in case of 0 .15m throat width, and 70% for wider throat widths up to 1m. Another advantage of a venturi-flume is, that: one control section can be installed for 2 to 3 grit chambers. Moreover, the venturi-flume is a self cleaning device, and there is no problem of clogging.

The discharge passing through a parshall flume of the type shown in Figure 3-6 is related to the water head *(i.e.* upstream water depth) by the formula:

$$Q = 2.264\, W * (H_a)^{3/2} \qquad (3.3)$$

Where, W = the width of the throat in m

Q = Flow in m^3/sec

H_a = Depth of flow in upstream leg of flume at one third point in m.

The above equation is applicable to flumes of 0.3m to 3m in width.

Typical design dimensions for parshall flumes are given in Table 3-1.

Table 3-1 Standard dimensions for parshall flumes (with respect to Figure 3-6) in cm

Flow range maximum in 10^6 l/sec	Throat width W	A	A_1	C	C_1	F	G	K	N
Up to 5	7.5	46.0	45.0	17.5	25.5	15.0	60.0	2.5	5.6
5 - 30	15.0	61.0	60.0	31.5	39.1	30.0	60.0	7.5	11.3
30 - 45	22.5	86.5	85.0	37.5	56.6	30.0	75.0	7.5	11.3
45 - 170	30.0	135.0	132.2	60.0	83.1	60.0	90.0	7.5	22.5
170 - 250	45.0	142.5	139.7	75.0	101.0	60.0	90.0	7.5	22.5
250 - 350	60.0	150.0	147.2	90.0	118.8	60.0	90.0	7.5	22.5
350 - 500	90.0	165.0	161.9	120.0	154.7	60.0	90.0	7.5	22.5
500 - 700	120.0	180.0	176.6	150.0	190.6	60.0	90.0	7.5	22.5
700 - 850	150.0	210.0	206.0	210.0	262.5	60.0	90.0	7.5	22.5
850 - 1400	240.0	240.0	235.3	270.0	334.4	60.0	90.0	7.5	22.5

ii. Parabolic Grit Channel

In order to maintain a constant horizontal flow velocity (V_h) through a grit channel, we have to ensure that the cross-sectional area of the channel changes with the changed discharge in direct proportion to the change in discharge. Thus, if x is taken along the width side and y is taken along the depth side of a channel x-section, then the cross-section must be such that

$$V_h = \frac{Q}{\int_0^y xdy} = \text{constant} = C_1 \tag{3.4}$$

Also, the discharge through the velocity control section, placed at the down end of the grit channel, in general, is given by the equation

$$Q = C_2 * y^n \tag{3.5}$$

Where, n is the discharge coefficient of the control section; 1.5 for venturi-flume, and 1 for proportional flow weir (also called Sutro weir).

Equating Q from equations (3.4) and (3.5), we get

$$C_1 \int_0^y xdy = C_2 * y^n$$

$$C_1 xy = C_2 * y^n$$

$$y^{n-1} = \frac{C_1 x}{C_2}$$

$$y^{n-1} = C' * x \tag{3.6}$$

For a parshall flume, n = 1.5,

$$y^{1.5-1} = C' * x$$

$$y = C * x^2 \quad (3.7)$$

Hence, when parshall flume is used as the velocity control device, then the channel section should be governed by equation $y = C * x^2$ (x representing width, and y representing depth); which is a parabolic section.

If a rectangular channel is used instead of a parabolic channel, then its width is constant:

i.e. x = k. Then, using equation (3.6),

$$y^{n-1} = C' * x = C' * k$$

$$y^{n-1} = C''$$

$$n - 1 = 0$$

$$n = 1$$

Hence, for a rectangular channel section, we need a control section, such as a Sutroweir, whose discharge equation is of the form $Q = K' * y$ where K is a constant.

Example 3-4

Design a grit chamber for a horizontal velocity of 25cm/sec and a flow which ranges from a minimum of 25000m^3/day to a maximum of 100,000m^3/day. Average flow is 62500m^3/day.

Solution

Let us adopt 4 grit channels, each designed to carry discharges as:

i. Minimum discharge $= \frac{25000}{4} = 6{,}250\text{cu m/day}$
ii. Normal maximum discharge $= \frac{100{,}000}{4} = 25{,}000\text{m}^3\text{/day}$
iii. Average discharge $= \frac{62500}{4} = 15{,}625\text{m}^3\text{/day}$
iv. Let each unit be designed to carry a peak discharge of 1.33 times the normal maximum discharge, $= 1.33 * 25000 = 33{,}325\text{m}^3\text{/day}$

Let a vertically controlled flume be used to maintain constant velocity. The flow in the control section is assumed to be at critical depth.

Let us now design the parabolic channel cross section:

For a parabolic channel,

Area of cross-section $= A = \frac{2}{3} B * D$

Where, B = Top width and D = Depth.

Since $A = \frac{Q}{V_h}$; and V_h = 0.25m/sec,

For all discharges, we can easily work out, ‘A’ values corresponding to peak, maximum, average, and minimum discharges.

Therefore,

$$\text{A for } Q_{peak}\ (A_{peak}) = \frac{Q_{peak}}{0.25}$$
$$= \frac{33325}{24 \times 60 \times 60} * \frac{1}{0.25} = \frac{0.386}{0.25} = 1.54\text{m}^2$$

Similarly,

$$\text{A for } Q_{max}\ (A_{max}) = \frac{Q_{max}}{0.25}$$
$$= \frac{25000}{24 \times 60 \times 60} * \frac{1}{0.25} = \frac{0.289}{0.25} = 1.16\text{m}^2$$

$$\text{A for } Q_{ave}\ (A_{ave}) = \frac{Q_{ave}}{0.25}$$
$$= \frac{15625}{24 \times 60 \times 60} * \frac{1}{0.25} = \frac{0.181}{0.25} = 0.72\text{m}^2$$

$$\text{A for } Q_{min}\ (A_{min}) = \frac{Q_{min}}{0.25}$$
$$= \frac{6250}{24 \times 60 \times 60} * \frac{1}{0.25} = \frac{0.072}{0.25} = 0.29\text{m}^2$$

At maximum discharge

Let us limit the maximum width of the channel to 1.5m at Q_{max}

Then,

$$A_{max} = \frac{2}{3} B * D_{max} = \frac{2}{3} * 1.5 * D_{max}$$

Equating eqn. *(ii)* and *(v)*, we have

$$1.16 = \frac{2}{3} * 1.5 * D_{max}$$

$$D_{max} = \frac{1.16 * 3}{2 * 1.5} = 1.16m$$

The total energy upstream of control section is given by:

$$E_1 = D + \frac{V_1^2}{2g}$$

(Neglecting velocity of approach in the channel)

$$E_1 = D$$

Hence at maximum discharge

$$E_1 = D_{max} = 1.16m$$

Total energy at critical point i.e. at the point of jump formation:

$$= E_c = y_c + \frac{V_c^2}{2g} + H_L$$

Where, H_L is the energy or head loss in jump

Assuming that the head loss in control section (H_L) is 10% of the velocity head in the control section, we have

$$H_L = 0.1\frac{v_c^2}{2g}$$

$$E_c = y_c + \frac{v_c^2}{2g} + 0.1\frac{v_c^2}{2g}$$

But, in the control section, at critical depth,

$$y_c = \frac{v_c^2}{g}$$

$$E_c = \frac{v_c^2}{g} + \frac{v_c^2}{2g} + 0.1\frac{v_c^2}{2g} = 1.55\frac{v_c^2}{g}$$

Using Bernoulli's theorem,

Total energy at upstream point in channel

= Total energy at critical point in control section

$$E_1 = E_c = 1.55\frac{v_c^2}{g}$$

From equation 6.9, $E_1 = D$

Hence,

$$D = 1.55\frac{v_c^2}{g}$$

Using the value of D_{max}, as equal to 1.16 at maximum discharge,

$$V_c \text{ at } Q_{max} = \sqrt{\frac{1.16 * 9.81}{1.55}} = 2.71 m/sec$$

$$y_c \text{ at } Q_{max} = \frac{{V_c}^2 \text{ at } Q_{max}}{g} = \frac{2.71^2}{9.81} = 0.74m$$

The discharge through the control section is:

$$Q = (W * y_c)\, V_c$$

Where, W is the throat width; and $W*y_c$ is the flow area of the throat.

$$W = \frac{Q}{y_c * V_c}$$

$$W = \frac{Q_{max}}{y_c * V_c(\text{both at } Q_{max})} = \frac{0.289}{0.74 * 2.71} = 0.144m \cong 0.15m$$

Let us use throat width W = 0.15m.

For other flow conditions:

Using the above used two formulas,

$$y_c = \frac{v_c^2}{g}$$

$$Q = W * y_c * V_c$$

$$V_c = \frac{Q}{W * y_c}$$

$$y_c = \frac{\left(\frac{Q}{W * y_c}\right)^2}{g}$$

$$y_c = \sqrt[3]{\frac{Q^2}{gW^2}}$$

Knowing Q and W, we can find y_c at different discharges.

From eqn. (9.13) derived above,

$$D = 1.55\frac{v_c^2}{g} = 1.55y_c$$

Knowing y_c at different discharges, we can find D at different discharges.

Then finally, for a parabolic section,

$$A = \frac{2}{3}B * D$$

$$B = 1.5\frac{A}{D}$$

Knowing A at various discharges, already computed, we can find B at different discharges, because D is known at different discharges.

The values of B are, thus, calculated for other discharges, as shown in table below.

Condition	Q m^3/day	Q in m^3/sec	A m^2	$y_c = \sqrt[3]{\frac{Q^2}{gW^2}}$ where W = 0.15	$D = 1.55y_c$	$B = 1.5\frac{A}{D}$
Peak	33,325	0.386	1.54	0.88	1.36	1.70*
Average	15625	0.181	0.72	0.53	0.82	1.32
Minimum	6250	0.072	0.29	0.29	0.45	0.97
Maximum	25000	0.289	1.16	0.72	1.12	1.56

*Limited to 1.5m.

With B and D values, computed in col. (6) and (7) of the above table, we can draw the parabolic section, which is approximated to a practical trapezoidal section, as shown in Figure 3-7.

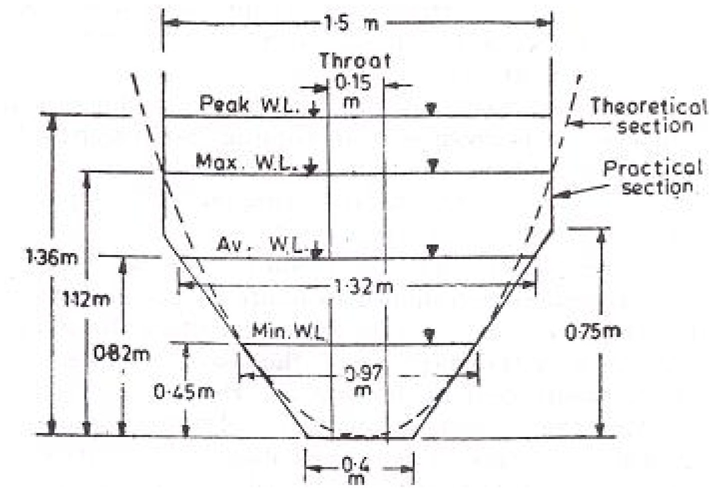


Figure 3-7 Grit channel section

3. Detritus Tanks

Detritus tanks are nothing but rectangular grit chambers, designed to flow with a smaller flow velocity (of about 0.09m/sec) and longer detention periods (about 3 to 4 minutes) so as to separate out not only the larger grit, etc., but also to separate out the very fine sand particles, etc. Due to this, a large amount of organic matter will also settle out along with the inorganic grit, sand, etc. This organic material is then separated from the grit by control of currents in the tank through baffles, or by controlled aeration of the flow through the tank. The rising air bubbles will then separate the lighter organic matter from the descending grit. The grit is removed continuously by means of scraper mechanism. All other details of detritus tanks remain the same as those of a rectangular grit chamber.

3.1.3 Tanks for Removing Oils and Grease

1. Skimming Tanks

Skimming tanks are sometimes employed for removing oils and grease from the sewage, and placed before the sedimentation-tanks. They are, therefore, used where sewage contains too much of grease or oils which include fats, waxes, soaps, fatty acids, etc. These materials may enter into the sewage from the kitchens of restaurants and, houses, from motor garages, oil refineries, soap and candle factories, etc. They are, thus, normally present in large amounts in the industrial wastewaters.

If such greasy and oily matter is not removed from the sewage before it enters further treatment units, it may form unsightly and odorous scums on the surface of the settling tanks, or interfere with the activated sludge treatment process, and inhibit biological growth on the trickling filters.

These oil and greasy materials may be removed in a skimming tank, in which air is blown by an aerating device through the bottom. The rising air tends to coagulate and congeal (solidify) the grease, and cause it to rise to the surface (being pushed in separate compartments), from where it is removed.

The typical details of a skimming tank are shown in Figure 3-8. It consists of a long trough shaped structure divided into two or three lateral compartments by means of vertical baffle walls (having slots in them) for a short distance below the sewage surface, as shown. The baffle walls help in pushing the rising coagulated greasy material into the side compartments (called stilling compartments). The rise of oils and grease is brought about by blowing compressed air into the sewage from diffusers placed at the bottom of the tank.

The collected greasy materials are removed (i.e. skimmed off either by hand or by some mechanical equipment. It may then be disposed of either by burning or burial.

Sewage enters the tank from one end, flows through longitudinally, and finally goes out through a narrow inclined duct, as shown. This is so narrow that the suspended heavier particles are carried up its slope and out of the tank. A detention period of about 3 to 5 minutes is usually sufficient, and the amount of compressed air required is about 300 to 6000m^3 per million liters of sewage. The surface area required for the tank can be found out by using the formula:

$$A = 0.00622 * \frac{q}{V_r} \tag{3.8}$$

Where, q = rate of flow of sewage in m^3/day

V_r = minimum rising velocity of greasy material to be removed in m/minute

= 0.25m/minute in most cases

The efficiency of a skimming tank can be increased considerably (three to four times) by passing chlorine gas (2mg/liter of sewage) along with the compressed air. Chlorine may also be added as a solution with the sewage discharge, just ahead of the air diffuser plates. The action of chlorine is to destroy the protective colloidal effect of protein, which holds the grease in emulsified form.

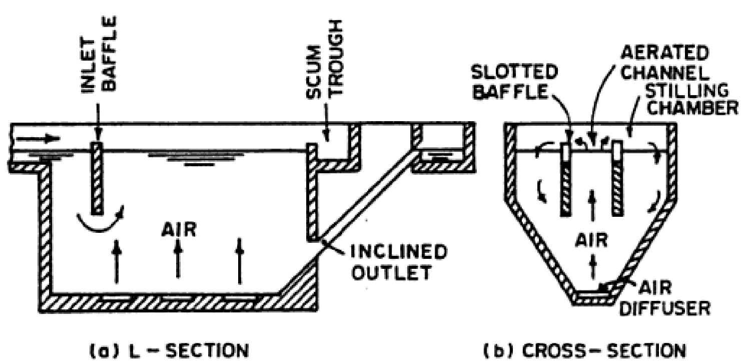


Figure 3-8 Skimming tank

2. Vacuators

Grease can also be removed from the sewage by vacuum floatation method, by subjecting the aerated sewage to a vacuum pressure of about 0 - 25cm of mercury for 10 to 15 minutes in a vacuator. This causes the air bubbles to expand and move upward through the sewage to the surface. The rising bubbles lift the grease and the lighter waste solids to the surface, where they are removed through skimming troughs. Heavier solids settle to the tank bottom, where they are collected and carried away for sludge treatment and disposal.

3. Disposal of Skimmings

The oil and greasy material removed as skimmings from the skimming tanks or vacuators can be disposed of either by burning or burial. It is generally too polluted to be of any economic use. However, it may sometimes be converted in to soap lubricants, candles and other non-edible products. It may sometimes be digested in digesters, which prove beneficial only if the mineral oils are less in amount, and vegetable and organic matters predominate. The latter digest easily, and produce gases of high fuel value.

3.2 Primary Wastewater Treatment

Primary treatment consists in removing large suspended organic solids. This is usually accomplished by sedimentation in settling basins.

The liquid effluent from primary treatment, often contains a large amount of suspended organic material, and has a high BOD (about 60% of original).

Sometimes, the preliminary as well as primary treatments are classified together, under primary treatment.

The organic solids which are separated out in the sedimentation tanks (in primary treatment) are often stabilized by anaerobic decomposition in a digestion tank or are incinerated. The residue is used for landfills or soil conditioners.

3.2.1 Sedimentation

1. Necessity of Sedimentation in Treatment of Wastewaters

As discussed in the previous pages, the screens and the grit chambers do remove most of the floating materials (like paper, rags, cloth, wood, tree branches, etc.) and the heavy inorganic settleable solids from the sewage. However, a part of the suspended organic solids which are too heavy to be removed as floating matters, and too light to be removed by grit chambers (designed to remove only the heavy inorganic solids of size more than 0.2 mm and of sp. gravity 2.65) are generally removed by the sedimentation tanks. The sedimentation tanks are thus designed to remove a part of the organic matter from the sewage effluent coming out from the grit chambers.

In a complete sewage treatment, the sedimentation is, in fact, carried out twice; once before the biological treatment (i.e. primary sedimentation) and once after the biological treatment (i.e. secondary sedimentation). When chemical coagulants are also used for flocculating the organic matter during the process of sedimentation, the process is called chemical precipitation or sedimentation aided with coagulation. This is generally not used in modern days, as discussed, later.

Other sewage treatment units which work on the principle of sedimentation are: Septic tanks, Imhoff tanks, etc. Septic tanks and Imhoff tanks combine sludge digestion with sedimentation, whereas the sludge deposited in primary as well as in the secondary settling tanks, is separately digested in the sludge-digestion tanks.

2. Types of Settling

Depending on the particles concentration and the interaction between particles, four types of settling can occur, see also Discrete Flocculent Hidered Compression

Figure 3-9:

1. **Discrete particle settling**

   The particles settle without interaction and occur under low solids concentration. A typical occurrence of this type of settling is the removal of sand particles.

2. **Flocculent settling**

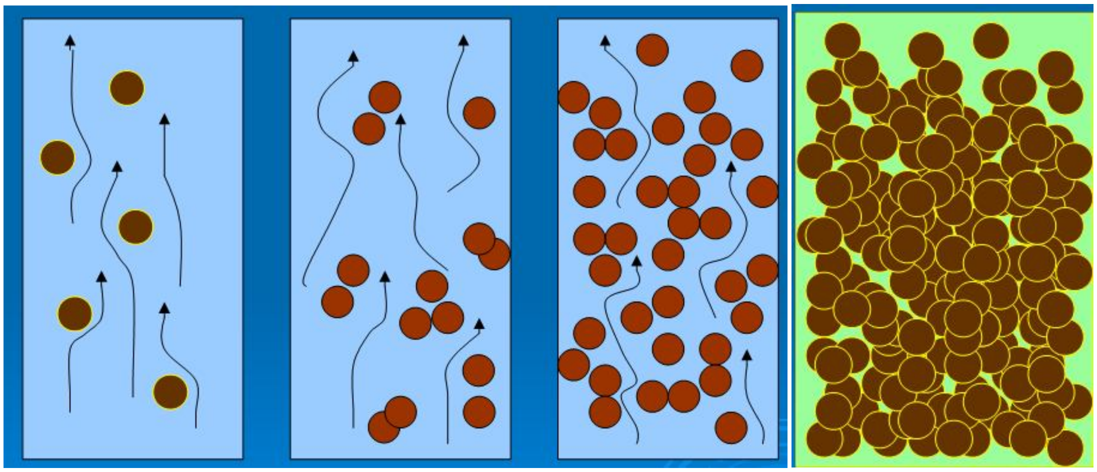
   This is defined as a condition where particles initially settle independently, but flocculate in the depth of the clarification unit. The velocity of settling particles is usually increasing as the particles aggregates. The mechanisms of flocculent settling are not well understood.

3. **Hindered/zone settling**

   Inter-particle forces are sufficient to hinder the settling of neighboring particles. The particles tend to remain in fixed positions with respect to each others. This type of settling is typical in the settler for the activated sludge process (secondary clarifier).

4. **Compression settling**

   This occurs when the particle concentration is so high that so that particles at one level are mechanically influenced by particles on lower levels. The settling velocity then drastically reduces.



Discrete Flocculent Hidered Compression

Figure 3-9 Settling phenomena in a clarifier

Settling of Discrete Particles (Type I Settling)

Sedimentation is the physical separation of suspended material from water or wastewater by the action of gravity. It is a common operation for water treatment and found in almost all wastewater treatment plants. It is less costly than many other treatment processes.

The very fundamental principle underlying the process of sedimentation is that the organic matter present in sewage is having specific gravity greater than that of water (i.e. 1.0). In still sewage, these particles will, therefore, tend to settle down by gravity; whereas, in flowing sewage, they are kept in suspension, because of the turbulence in water. Hence, as soon as the turbulence is retarded by offering storage to sewage, these impurities tend to settle down at the bottom of the tank offering such storage. This is the principle behind sedimentation.

The design of sedimentation basins is thus totally dependent upon the *settling velocity* of the sewage solids. The design of an ideal settling basin is based on the removal of all particles that have a settling velocity greater than a specified settling velocity.

The settling velocity of a discrete solid particle is mathematically computed and analyzed on the basis of the theory of sedimentation, which is discussed below:

i. **Theory of Type I Settling**

The settlement of a particle in water brought to rest is opposed by the following factors:

(i) The velocity of flow which carries the particle horizontally.
The greater the flow area, the lesser is the velocity, and hence more easily the particle will settle down.

(ii) The viscosity of water in which the particle is travelling.

The viscosity varies inversely with temperature. Warm water is less viscous and, therefore, offers less resistance to settlement. However, the temperature of water cannot be controlled to any appreciable extent in "water or wastewater purification processes" and hence this factor is generally ignored.

(iii) The size, shape and specific gravity of the particle.

The greater is the specific gravity, more readily the particle will settle. The size and shape of the particle also affect the settling rate. For example, the weight and volume of the spherically shaped particle varies with the cube of its diameter (volume being equal to $\frac{\pi d^3}{6}$, where d is the diameter) or its size; and its area varies with the square of the diameter (area being equal to $\frac{\pi d^2}{4}$). Hence, very small sized particles will settle very slowly. It, therefore, clearly follows that the shape and size of the particles do affect their setting velocities.

The settling velocity of a spherical particle is expressed by **Stoke's law**, which takes the above three factors into account. The final Stoke's equation for d < 0.1mm is expressed as:

$$v_s = \frac{g}{18}(G-1)\frac{d^2}{\upsilon} \qquad (3.9)$$

[For viscous flow and small sized particles represented by Re < 1]

Where, v_s = velocity of settlement of particle (assumed to be spherical) in m/sec

d = diameter of the particle in m

G = sp. gravity of the particle

υ = kinematic viscosity of water in m^2/sec and is equal to $\frac{\mu}{\rho_w}$

Where, μ = absolute or dynamic viscosity of water in kg.sec/m^2

ρ_w = density of water

Derivation of Stoke's Law

When a solid particle settles down in water, its downward settlement is opposed by the drag force offered by the water. The effective weight of the particle (i.e. actual weight buoyancy) causes the particle to accelerate in the beginning, till it attains a sufficient velocity (v_s) at which the drag force becomes equal to the effective weight of the particle. After attaining that velocity, the particle falls down with that constant velocity (v_s).

Now, the drag force offered by the fluid is given by Newton's law, as

$$\text{Drag force} = C_D A * \rho_w * \frac{v^2}{2} \qquad (3.10)$$

Where, C_D = Coefficient of drag

A = Area of particle

ρ_w = Density of water

v = velocity of fall

Note: This drag force increases with the increasing velocity till it becomes equal to the effective weight of the particle; and at that time, v becomes equal to v_s.

The effective weight of the particle

$$= \text{Total weight} - \text{Buoyancy}$$

$$= \frac{4}{3}\pi r^3 \gamma_s - \frac{4}{3}\pi r^3 \gamma_w = \frac{4}{3}\pi r^3(\gamma_s - \gamma_w) \tag{3.11}$$

Where, r = radius of particle

γ_s = unit weight of particle

γ_w = unit weight of water

Eqs. (3.10) and (3.11) will become equal when v becomes equal to v_s in Eq. (3.10).

$$C_D\, A * \rho_w * \frac{{v_s}^2}{2} = \frac{4}{3}\pi r^3(\gamma_s - \gamma_w)$$

$$C_D\, \pi r^2 * \rho_w * \frac{{v_s}^2}{2} = \frac{4}{3}\pi r^3(\gamma_s - \gamma_w)$$

$${v_s}^2 = \frac{\frac{8}{3} * (\gamma_s - \gamma_w)\frac{d}{2}}{C_D * \rho_w}$$

$${v_s}^2 = \frac{4}{3}\frac{* g(\rho_s - \rho_w)d}{C_D * \rho_w}$$

$${v_s}^2 = \frac{4}{3}\frac{* g(G-1)d}{C_D} \tag{3.12}$$

The coefficient of drag (C_D) has been found for a viscous flow and small particles (size d < 0.1mm) to be equal to $\frac{24}{R_e}$

Where, R_e is the particle Reynolds number $= \frac{v_s d}{\upsilon}$

Therefore, Eq. (3.12) then becomes,

$${v_s}^2 = \frac{4}{3}\frac{* g(G-1)d}{\frac{24}{R_e}}$$

$${v_s}^2 = \frac{4}{3} * g(G-1)d * \frac{R_e}{24}$$

$${v_s}^2 = \frac{g}{18} * (G-1)d * \frac{v_s d}{\upsilon}$$

$$v_s = \frac{g}{18} * (G-1) * \frac{d^2}{\upsilon} \tag{3.13}$$

The above Stoke's equation is valid for particles of size less than 0.1mm; in which case, the viscous force predominates over the inertial force, leading to what is known as streamline settling.

If, however, the settling particles are larger than 0.1mm, the nature of settling tends to become turbulent, with a transition zone in between. It has been established that turbulent settling occurs for particle size greater than 1.0mm, whereas settling remains transition settling for particle sizes between 0.1mm to 1.0mm.

The relation between coefficient of drag (C_D) and R_e for these three types of settling are as follows:

(a) For streamline settling (d < 0.1mm) Here $R_e < 1$; and

$$C_D = \frac{24}{R_e} \qquad (3.14)$$

(b) For transition settling (d between 0.1mm and 1.0mm)

Here $1 < R_e < 10^3$

$$C_D = \frac{24}{R_e} + \frac{3}{\sqrt{R_e}} + 0.34 \qquad (3.15)$$

(c) For turbulent settling (d > 1.0mm)

Here $R_e < 10^3$

$$C_D = 0.34 \text{ to } 0.4$$

For turbulent settling, the equation 3.12 reduces to:

$$v_s = 1.8\sqrt{gd * (G - 1)} \qquad (3.16)$$

This equation is known as Newton's equation for turbulent settling.

The above formulas represent the theoretical settling velocities of discrete spherical particles. The actual settling velocities in the sedimentation basins will be much less than those calculated by these formulas, because of the non-sphericity of the particles, the upward displacement of the fluid caused by the settling of other particles, and convection currents.

Based upon experiments, Hazen has formulated a table (Table 3-2) giving the values of the settling velocities (popularly called hydraulic settling values) for different sized particles in still liquids at 10^oc.

Table 3-2 Hydraulic settling values in mm/sec in still liquids at 10ºc

Type of material	Diameter of particles in mm	Settling velocity in mm/sec	
		Particles of Sp. gravity 2.65	Particles of Sp. gravity 1.20
Course sand	1.00	100	12.0
	0.80	83	9.6
	0.60	63	7.2
Medium sand	0.50	53	6.0
	0.40	42	4.8
	0.30	32	3.6
Fine sand	0.20	21	2.4
	0.15	15	1.8
	0.10	8	1.2
Very fine and	0.08	6	0.53
	0.06	3.8	0.30
	0.05	2.9	0.21
Silt	0.04	2.1	0.13
	0.03	1.3	0.076
	0.02	0.62	0.034
	0.015	0.35	0.019
	0.010	0.154	0.0084
Fine silt	0.008	0.098	0.0054
	0.006	0.055	0.0030
	0.005	0.0385	0.0021
Clay	0.004	0.0247	0.0013
	0.003	0.0138	0.00076
	0.002	0.0062	0.00034
	0.0015	0.0035	0.00019
Fine clay	0.001	0.00154	0.000084
	0.0001	0.0000154	0.00000084
Colloidal clay	0.0001		

At higher temperatures such as 26°c (i.e. average temperature prevailing in our country), the value of settling velocity will be about 50% more than these values.

The above experimental values have also been expressed in mathematical form as modified Hazen's equation for transition zone, given by:

$$v_s = 60.6d * (G - 1) * \left(\frac{3T + 70}{100}\right) \tag{3.17}$$

For particles between 0.1 and 1mm, the above equation yields the following:

For inorganic solids, G = 2.65;

Settling velocity for inorganic solids

$$v_{s(in)} = 60.6d * (1.65) * \left(\frac{3T + 70}{100}\right)$$

$$v_{s(in)} = d * (3T + 70) \qquad (3.18)$$

Similarly, settling velocity for organic matter (for which G = 1.2)

$$v_{s(or)} = 60.6d * (10.2) * \left(\frac{3T + 70}{100}\right)$$

$$v_{s(or)} = 0.12d * (3T + 70) \qquad (3.19)$$

Example 3-5

Estimate the settling velocity in wastewater at a temperature of 25°c of spherical silicon particles with specific gravity 2.67 and average diameter of:

(i) 0.04mm Answer: **0.162cm/sec**

(ii) 0.4mm Answer: **7.56cm/sec**

3. Sedimentation Tanks

The clarification of sewage by the process of sedimentation can be affected by providing conditions under which the suspended material present in sewage can settle out. This is brought about in specially designed tanks called ***sedimentation tanks***.

Out of the three forces which control the settling tendencies of the particles, the two forces i.e. the velocity of flow and the shape and size of the particles are tried to be controlled in these settling tanks. The third force i.e. the viscosity of sewage or the temperature of sewage is left uncontrolled as the same is not practically possible.

The velocity of flow can be reduced by increasing the length of travel and by detaining the particle for a longer time in the sedimentation basin. The size and the shape of the particles can be altered by the addition of certain chemicals in water. These chemicals are called coagulants, and they make the sedimentation quite effective leading to the settlement of even very fine and colloidal particles. However, their use is not made in plain sedimentation (or generally called sedimentation) but is being made in the process called chemical precipitation or sedimentation with coagulation.

Sedimentation basins are thus designed for effecting settlement of particles by reducing the flow velocity or by detaining the sewage in them. They are generally made of reinforced concrete and may be rectangular or circular in plan. Long narrow rectangular tanks with horizontal flow (Figure 3-10) are generally preferred to the circular tanks with radial or spiral flow (Figure 3-11).

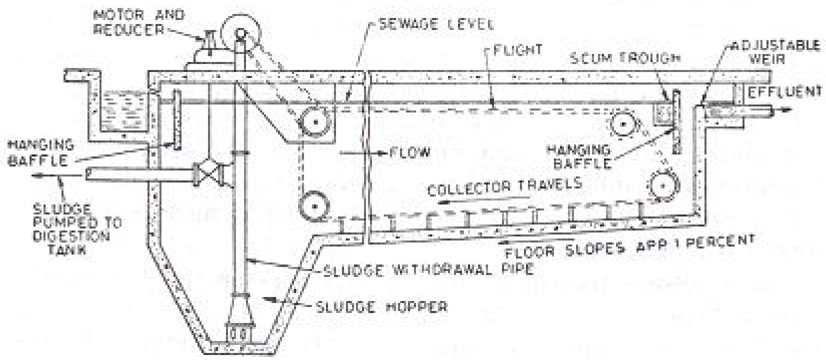


Figure 3-10 Rectangular sedimentation tank

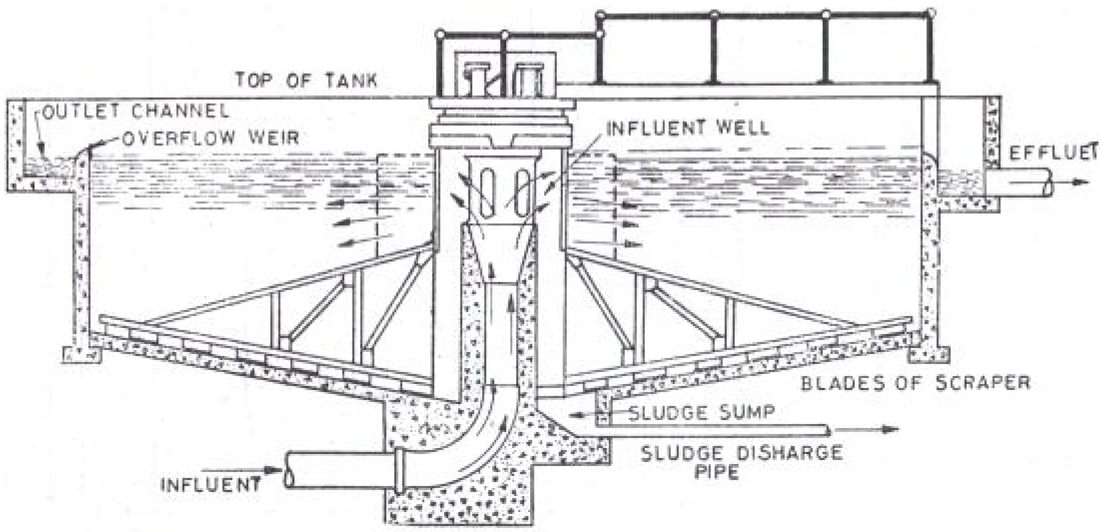


Figure 3-11 Circular sedimentation tank

The capacity and other dimensions of the tank should be properly designed, so as to affect a fairly high percentage of removal of the suspended organic material. A plain sedimentation tank under normal conditions may remove about 60 to 65% of the suspended solids, and 30 to 35% of the BOD from the sewage.

i. Types of Sedimentation Tanks

Sedimentation tanks may function either intermittently or continuously.

The **Intermittent settling** tanks called quiescent type tanks are simple settling tanks which store sewage for a certain period and keep it in complete rest. After giving it a rest of about 24hours, during which the suspended particles settle down to the bottom of the tank, the cleaner sewage from the top may be drawn off and the tank be cleaned off the settled silt. The tank is again filled with raw sewage to continue the next operation. This type of tank, thus, functions intermittently as a period of about 30 to 36 hours is required to put the tank again in working condition. This necessitates the commissioning of at least two tanks. Such tanks are generally not preferred, because a lot of time and labor is wasted and more units are required. They have, therefore, become comp1etely obsolete these days.

In a **continuous flow** type of a sedimentation tank, which is generally used in modem days, the flow velocity is only reduced, and the sewage is not brought to complete rest, as is done in an intermittent type. The working of such a tank is simple, as the wastewater enters from one end, and comes out from the other end. The velocity is sufficiently reduced by providing sufficient length of travel. The velocity is so adjusted that the time taken by the particle to travel from one end to another is slightly more than the time required for settlement of that particle. The theory and design of such a tank is discussed below in details.

ii. Design of a Continuous Flow Type of a Sedimentation Tank

In the theory which is applied to the design of such sedimentation basins, it is assumed that the sediment is uniformly distributed as the sewage enters the basin. In Figure 3-12, let the wastewater containing uniformly distributed sediment enters the rectangular tank with a uniform velocity V. If Q is the discharge entering the basin, the flow velocity V is given by:

$$V = \frac{Q}{BH} \tag{3.20}$$

Where, B = Width of the basin, and

H = Depth of sewage in the tank

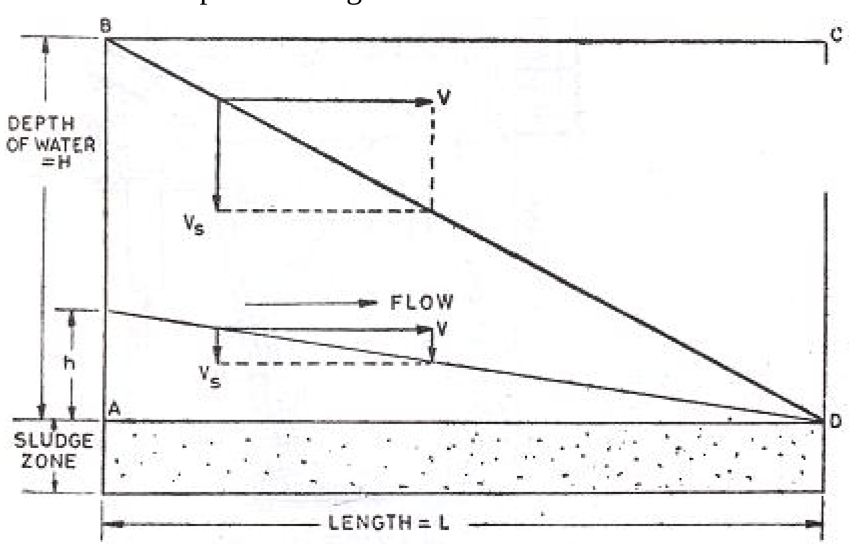


Figure 3-12 Elevation of a rectangular sedimentation tank

In the settling zone, every discrete particle is moving with a horizontal velocity V and a downward vertical velocity V_s. The resultant path is given by the vector sum of its flow velocity (V) and its settling velocity (V_s).

Assuming that all those particles, whose paths of travel are above the line BD, will pass through the basin, we have from geometric considerations:

$$\frac{V}{V_s} = \frac{L}{H}$$

$$V_s = \frac{V * H}{L}$$

$$V_s = \frac{H}{L} * \frac{Q}{BH} = \frac{Q}{BL} \text{ ... ... overflow rate} \qquad (3.21)$$

It shows that all those particles with a settling velocity equal to or greater than $\frac{Q}{BL}$ will settle down and be removed. In other words, no particle having a settling velocity more than or equal to $\frac{Q}{BL}$ will remain suspended in such a tank.

It was mentioned above, that a particle having settling velocity greater than or equal to $\frac{Q}{BL}$ will be removed. In fact, it was the case when the particle entering at full height H of the tank was considered. Truly speaking, even the smaller particles having settling velocities (V_s') lower than $\frac{Q}{BL}$ will also settle down, if they happen to enter at some other height h of the tank. In that case, when the particles are entering at some other height h of the tank, all those particles having their settling velocities $\geq \frac{h}{H}\frac{Q}{BL}$ will settle down.

The ratio of removal of this size particle to that of settling value V_s is given by:

$$X_r = \frac{h}{H} = \frac{V_s'}{V_s} \qquad (3.22)$$

Where, X_r – the ratio of removal of the given size particle

V_s' - the settling velocity of the given size particle

V_s - the settling velocity of the design/stated particle

For example:

Suppose the tank has 400m^2 surface area and ratio of inflow is 1.6m^3/s, then $\frac{Q}{A} = \frac{1.6}{400} = 0.4\text{cm/s}$

Hence,

- All particles having $V_s = 0.4\text{cm/sec}$ will be removed
- Only 50 % of the particles having $V_s = 0.2\text{cm/sec}$ will be removed
- Only 25% of the particles having $V_s = 0.1\text{cm/sec}$ will be removed and so on.

It, therefore, follows that the quantity $\frac{Q}{BL}$ i.e. the discharge per unit of plan area is a very important term for the design of continuous flow type of settling tanks; and is known as ***overflow rate or surface loading or overflow velocity***.

The efficiency of a sedimentation tank indicates the overall percentage removal of suspended matter at a given overflow rate V_s.

Prediction of efficiency of basin requires a settling column analysis from which the cumulative frequency distribution curve may be obtained Figure 3-13.

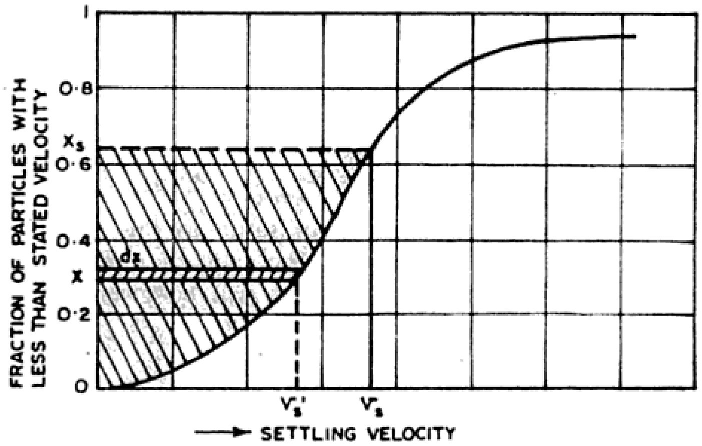


Figure 3-13 Cumulative particles removal versus settling velocity curve

According to the equation $X_r = \frac{h}{H} = \frac{V_s'}{V_s}$, the weight fraction removal of particles having velocity $V_s' < V_s$ will be:

$$= \int_0^{x_s} \frac{V_s'}{V_s} dx = \frac{1}{V_s} \int_0^{x_s} V_s' dx \tag{3.23}$$

Hence, the overall fraction of particles removed, F, would be:

$$F = (1 - x_s) + \frac{1}{V_s} \int_0^{x_s} V_s' dx \tag{3.24}$$

Approximation:

$$F = (1 - x_s) + \frac{1}{V_s} \sum V_s' \Delta x \tag{3.25}$$

In which $(1 - x_s)$ is the fraction with $V_s' \geq V_s$.

Normal values of overflow rates V_s range between:

- 40,000 and 50,000liters/m^2/day for plain primary sedimentation tanks
- 50,000 and 60,000liters/m^2/day for sedimentation tanks using coagulants as aids
- 25,000 to 35,000liters/m^2/day for secondary sedimentation tanks

Decreasing the overflow rate will lead to the settlement of even those particles which are having lower values of their settling velocities. Hence, smaller particles will also settle down, if the overflow rate is reduced. Further, with a given Q, the overflow rate can be reduced by increasing the plan area of the basin. It therefore, follows that an increase in the plan area (i.e. width x length) will increase the efficiency of sedimentation tank; and theoretically speaking, depth does not have any effect on the efficiency of sediment removal. However, it is important for practical considerations, and also for making allowance for deposition of sludge and silt.

Usual values of effective depth (i.e. depth excluding the bottom sludge zone) range between 2.4 and 3.6m (generally not exceeding 3m).

Another important term, which is used in connection with the design of sedimentation basins, is its detention time or detention period or retention period. The detention time (t) of a settling tank may be defined as the average theoretical time required for the sewage to flow through the tank. It is, this, the time that would be required for the flow of sewage to fill the tank, if there was no outflow. In other words, it is the average time for which the sewage is detained in the tank. Hence, it is the ratio of the volume of the basin to the rate of flow (i.e. discharge) through the basin.

Therefore,

Detention time, t, for a rectangular tank

$$t = \frac{\text{Volume of the tank}}{\text{Rate of flow}} = \frac{B * L * H}{Q} = \frac{L}{V} \qquad (3.26)$$

Similarly, the detention time for a circular tank

$$t = \frac{d^2(0.011d + 0.785H)}{Q} \qquad (3.27)$$

Where, d = Diameter of the tank

H = Vertical depth at wall or side water depth.

The detention time for a sewage sedimentation tank usually ranges between 1 to 2 hours. The lower value of detention period (i.e. 1 hour) is generally adopted when the activated sludge treatment is used in secondary treatment after the sedimentation; and the higher and more normal value (i.e. 2 hours) is generally adopted when the trickling filters are used as the secondary treatment.

Larger detention periods will result in higher efficiency; but too long a period induces septic conditions, and should be avoided. However, if the secondary sedimentation is to be avoided, a longer detention period of about $2\frac{1}{2}$ hours to 3 hours may be adopted.

The width of the tank is normally kept at about 6m, and not allowed to exceed 7.5m. The length of the tank is generally not allowed to exceed 4 to 5 times the width. The cross-sectional area of the sedimentation tank is such as to provide a horizontal flow velocity of about 0.3m/minute. The total amount of flow from the tank within 24 hours generally equals the maximum daily flow of sewage.

The maximum diameter of a circular tank may be kept 60m or so.

Example 3-6

A clarifier is designed to have a surface overflow rate of 28.53 $m^3/m^2/d$. Estimate the overall removal with the settling analysis data and particle size distribution in cols. 1 and 2 of Table 1. The wastewater temperature is 15°c and the specific gravity of the particles is 1.20.

Table 1: Results of settling analysis test and estimation of overall solid removal

Particle size in mm	Weight fraction < size, %	Settling velocity in mm/sec
0.1	12	0.968
0.08	18	0.620
0.07	35	0.475
0.06	72	0.349
0.05	86	0.242
0.04	94	0.155
0.02	99	0.039
0.01	100	0.010

Settling velocities versus cumulative distribution curve is given in the figure below.

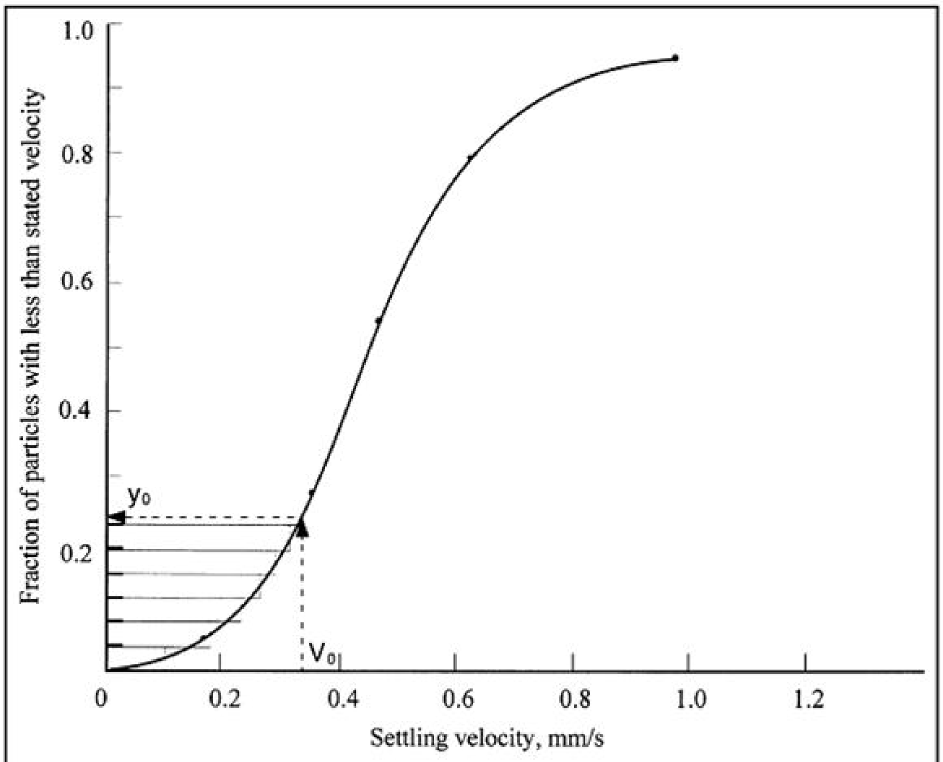


FIGURE Cumulative particles removal versus settling velocity curve

Referring the figure:

Δy	0.04	0.04	0.04	0.04	0.04	0.04	0.01
V	0.09	0.17	0.23	0.26	0.28	0.31	0.33
$V\Delta y$	0.0036	0.0068	0.0092	0.0104	0.0112	0.0124	0.0033

Answer: F = 0.92 = 92%

iii. Short Circuiting in the Sedimentation Tanks

For the efficient removal of sediment in the sedimentation tanks, it is necessary that the flow is uniformly distributed throughout the cross-section of the tank. If currents, on the other hand, permit a substantial portion of the water to pass directly through the tank without being detained for the intended time, the flow is said to be short circuited. Properly designed inlets and outlets near the entrance and the exit may reduce the short circuiting tendencies, and distribute the flow more evenly. Moreover, relatively narrow tanks are less affected by inlet and outlet disturbances, and by currents caused by breezes.

But however, in actual practice, certain amount of short circuiting will always exist, and, therefore, the actual average time taken by a batch of water in passing through a settling tank (called flowing through period) will always be less than the detention period, which is the corresponding theoretical time. The ratio of the 'flowing through period' to the 'detention period' is called the Displacement efficiency.

$$\text{Displacement efficiency (\%)} = \frac{\text{Flowing through period}}{\text{Detention period}} \qquad (3.28)$$

Note: In order to counteract the effects of short circuiting, it may be necessary to keep a high detention period or a smaller surface loading than that obtained from the theoretical considerations for obtaining the desired results.

iv. Constructional Details of the Sedimentation Tanks

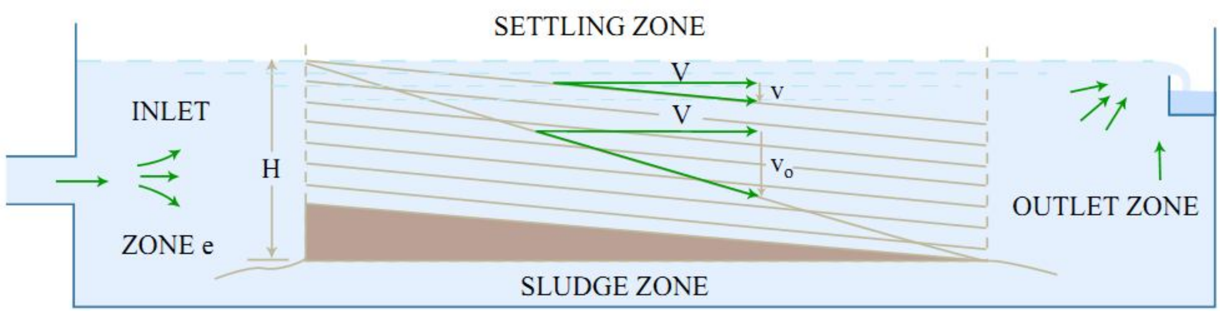
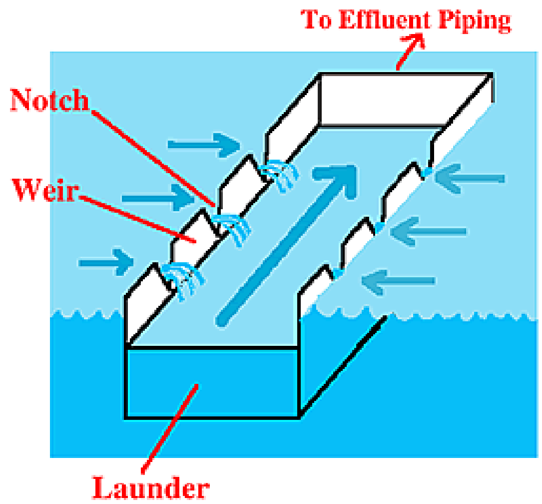


Figure 3-14 Zones of a rectangular horizontal continuous flow sedimentation tank

a. Inlet and Outlet Arrangement

In order to reduce short circuiting and to distribute the flow uniformly proper arrangement must be made for smooth entry of water. A most suitable type of an inlet for a rectangular settling tank is in the form of a channel extending to full width of the tank, with a submerged weir type baffle wall, as shown in Figure 3-16.



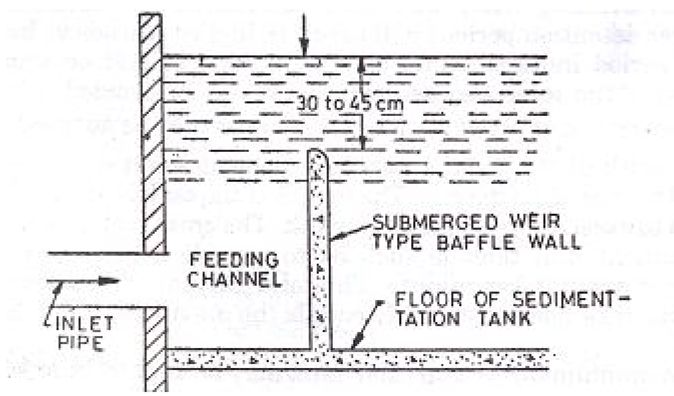
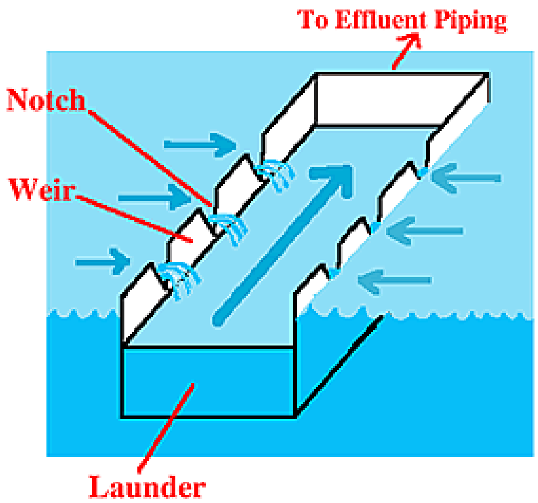


Figure 3-15 Section of a submerged type or a weir type inlet

A similar type of outlet arrangement is also used these days. It consists of an outlet channel extending for full width of the tank and receiving the water after it has passed over a weir, as



shown in

Figure 3-16.

Outlet arrangement consists of

i. weir, notches or orifices
ii. effluent trough or launder
iii. outlet pipe

Weir loading: The outlet weirs drawn off wastewater and take it out without disturbing the quiescent conditions in the tank. Weir loading influences the removal of solids in sedimentation tank, particularly in secondary settling tanks where flocculated solids are settled. For all primary settling tanks weir loading should not be greater than $100m^3/d/m$. for average flow is recommended. For secondary settling tanks in activated sludge process, weir loading should not be greater than $150m^3/d/m$. performance of sedimentation tanks can be improved by merely increasing their weir length.

Weir loading rates are limited to prevent high approach velocities near the outlet. Weirs frequently consist of V-notches approximately 50mm in depth, placed 150 – 300mm on centers, with a baffle in front of the weir to prevent floating material from escaping the sedimentation basin and clogging the filters.

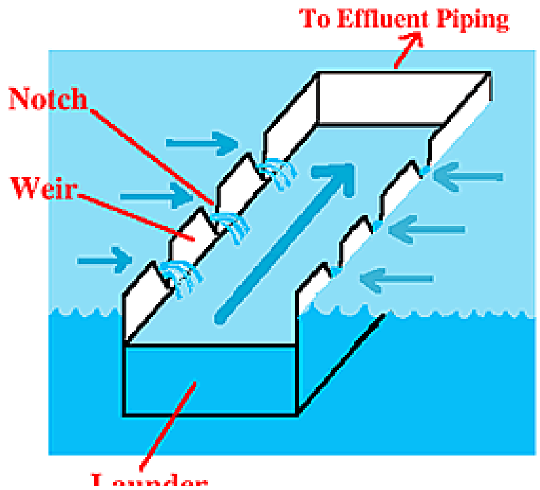


Figure 3-16 Weir type outlet

b. Baffles

Baffles are required to prevent the movement of organic matter and its escape along with the effluent; and to distribute the sewage uniformly through the cross-section of the tank, and thus to avoid short circuiting. Both inlets and outlets are, therefore, protected by hanging baffles, 0 to 90 cm in front of them, and submerged 45 to 60 cm below the flow line, as shown in Figure 3-10. Some other arrangement of placing baffles may be designed, but it should not be too complicated. Moreover, too many baffles may result in concentration of currents and is as bad as too fewer baffles are.

c. Skimming Troughs

When the amount of oils and greasy matter present in sewage is small, it is generally uneconomical to provide a separate skimming tank. In such cases, a skim trough is generally provided in the sedimentation tank itself, near its outlet end, as shown in Figure 3-10.

In manually operated tanks, the skimming that float to the surface may be pushed into the trough by squeezes with hand. Whereas, in mechanically operated tanks (such as in Figure 3-10), the skimming may be pushed by the same scraper blades which collect the sludge while moving along the bottom, and push the skimming into the end trough, when they move near the surface along with the endless chain to which they are attached.

d. Cleaning and Sludge Removal

The suspended organic solids contained in sewage, settle down at the bottom of the sedimentation tank, and have to be removed periodically. The removal of the deposited sludge before it becomes stale and septic is necessary not only because it reduces the capacity of the tank and its detention period, but also because it leads to the evolution of foul gases formed due to the anaerobic decomposition of the settled organic matter. The sedimentation tanks are, therefore, cleaned from time to time at frequent intervals, either manually or they are provided with mechanical arrangements for cleaning.

Modern sedimentation tanks, however, are generally provided with mechanical cleaning devices. For example, in Figure 3-10, the sludge is scraped by scrapers and brought to the hopper at the outlet end, and is removed daily or often. The scrapers can work either continuously or at any desired intervals of time. Similarly, in a circular tank (Figure 3-11), the sludge is scraped and brought to the centre, and likewise removed. For tanks without mechanical sludge removing equipment, an additional minimum depth of about 0.8 to 1.2 m should be provided for storage of settled materials, and is called sludge zone.

Example 3-7

Design a suitable rectangular sedimentation tank (provided with mechanical cleaning equipment) for treating the sewage from a city provided with an assured public water supply system, with a maximum daily demand of 12 million liters per day. Assume suitable values of detention period and velocity of flow in the tank. Make any other assumptions, wherever needed.

3.2.2 Sedimentation Aided with Coagulation (Type II Sedimentation)

1. Chemical Precipitation and Coagulation

Very fine suspend particles, present in wastewaters, which cannot be removed in plain sedimentation, may sometimes, be settled by increasing their size be changing them into flocculated particles. For this purpose, certain chemical compounds (like ferric chloride, ferric sulphate, alum, chlorinated copperas, etc.) called coagulants are added to the wastewaters, which on thorough mixing form a gelatinous precipitate called floc. The fine mud particles and other colloidal matter present in wastewaters get absorbed in these floes forming the bigger sized flocculated particles. The process of addition and mixing of chemicals is called coagulation. The coagulated sewage is then made to pass through sedimentation tank where the flocculated particles settle down and get removed.

The characteristics and efficiency of the important coagulants used in sewage treatment are given in Table 3-3.

Table 3-3 Properties of the important coagulants used in sewage treatment

.No.	Name of coagulant	B.O.D. removed as percentage of total present	SS removed as percentage of total present	Dosage required in ppm	pH value required for proper functioning	Remarks
1.	Ferric chloride	80 - 90	90 - 95	25 - 35	5.5 to 7.0	This coagulant is widely used for sewage treatment, wherever, coagulation is adopted.
2.	Ferric sulphate with lime	60	80	35 - 40	8.0 to 8.5	Ferric sulphate has been found to be more effective than chlorinated copperas, if used in conjunction with lime. Hence ferric chloride and ferric sulphate are mainly used, as coagulants in sewage.
3.	Alum	60	80	40 - 90	6 to 8.5	It is generally not used in sewage although used for treating water supplies on a large scale.
4.	Chlorinated copperas	70 - 80	80 - 90	35 - 80	5.5 to 7.0 and 9.0 to 9.5	This coagulant is effective for producing sludge for activated sludge process.

2. Merits and Demerits of Coagulation Process in Sewage Treatment

As pointed out earlier, the coagulation process is generally not adopted in modern sewage treatment plants, mainly because of the following reasons:

1. More advanced methods of sewage treatment based on biological actions are available these days, and they are preferred to coagulation.
2. The coagulation process has various disadvantages, such as discussed below:
   (i) The biological secondary treatments used these days for treating sewage is complete in themselves, and do not require coagulation. Moreover, coagulation rather makes some of these processes more difficult.
   (ii) The chemicals used in coagulation react with sewage, and during these reactions, they destroy certain micro-organisms, which are helpful in digestion of the sludge, thus creating difficulties in sludge digestion.
   (iii) Cost of chemicals is added to the cost of sedimentation, without much use, and thereby making the treatment costlier.

(iv) The process of coagulation and subsequent sedimentation produces larger quantities of sludge than that produced in plain sedimentation, and thus adding to the problems of sludge disposal.

(v) The process of coagulation requires skilled supervision and handling of chemicals.

In view of all these disadvantages, the coagulation of sewage has become obsolete these days. It may still, however, be adopted in certain special cases, such as:

(a) For treating sewage from industries, using some specific chemicals in their processes.

(b) It is particularly advantageous, where there is large seasonal variation in sewage flow or as an emergency measure to increase the capacity of an overloaded plain sedimentation tank.

Table 3-4 Summery of preliminary and primary treatment unit functions and efficiencies

S.No	Type of Treatment	Purification effected	Process or unit employed	BOD removal as percentage of original	Removal of SS and DS as percentage of original	Removal of Bacterial load as percentage of original	Disposal of residuals
1.	Preliminary Treatment	(a) Removal of floating materials dead animals, like tree branches, pieces of rags, wood and other large sized floating materials.	Coarse and fine screens of different designs	5 - 10	2 - 20	10 - 20	Screenings can be disposed of easily, either by burials or burnings.
		(b) Removal of heavy settleable inorganic solids.	Grit chambers or Detritus tanks	10 - 20	20 - 40	10 - 20	The grit can be easily disposed of either by burials or burnings for raising low lying areas.
		(c) Removal of fats and greases	Skimming tanks or Vacuators	20 - 30	20 - 40	10 - 20	The skimming contains unstable volatile organic materials and have to disposed of by first stabilizing them in digestion tanks by anaerobic process.
2	Primary Treatment	Removal of suspended settleable organic solids	(i) Sedimentation tanks or	30 - 35	60 - 65	25 - 75	Sludge containing organic material has to be stabilized first, in digestion tanks and the digested material is then used as a manure or soil builder.

4- SECONDARY/BIOLOGICAL AND TERTIARY WASTEWATER TREATMENT

4.1 The Role of Microorganisms in Wastewater Treatment

Micro-organisms, such as bacteria, play an important role in the natural cycling of materials and particularly in the decomposition of organic wastes. The role of micro-organisms is elaborated further here because they are also important in the treatment of wastewater. Waste form humans become a useful food substrate for the micro-organisms. In both natural and engineered treatment systems micro-organisms such as bacteria, fungi, protozoa, and crustaceans play an essential role in the conversion of organic waste to more stable, less polluting substances. They form what is termed a 'food chain'.

In a natural water body, e.g. river or lake, the number and type of micro-organisms depends on the degree of pollution. The general effect of pollution appears to be a reduction in species numbers. For example in a badly polluted lake, there are fewer species but in larger numbers, while in a healthy lake there can be many species present but in lower numbers.

Micro-organisms are always present in the environment and given the right conditions of food availability, temperature and other environmental factors, they grow and multiply (Figure 4-1)

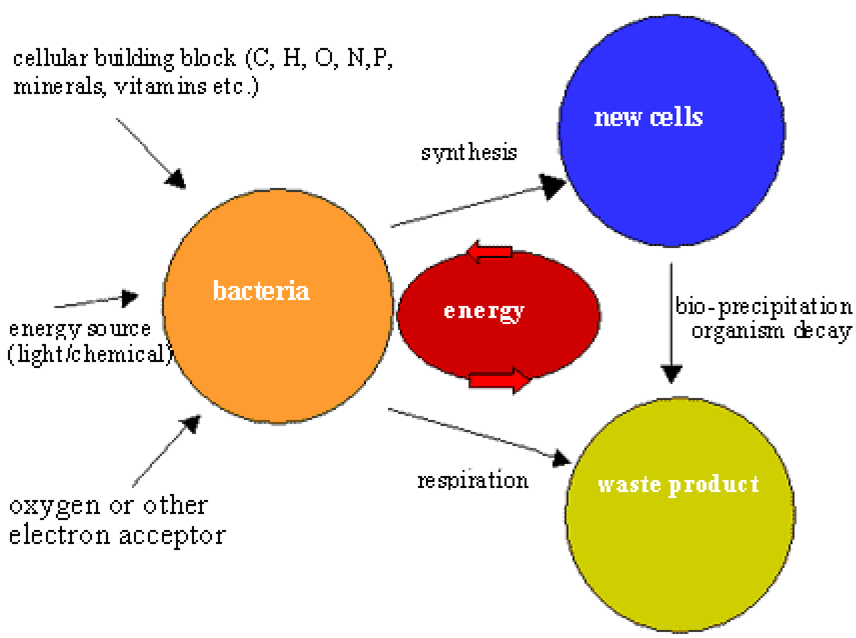


Figure 4-1 Generalized representation of growth of micro-organisms

Micro-organisms require cellular building blocks, such as (carbon) C, (hydrogen) H, (oxygen) O, (nitrogen) N, (phosphorus) P, and minerals for growth. These can be obtained through consuming organic substances containing these elements, or from inorganic materials, such as carbon dioxide, water, nitrate and phosphate. Micro-organisms also require energy. They obtain this through respiration. In this process organic carbon is oxidized to release its energy. Oxygen or other hydrogen acceptors is needed for the respiration process. Algae and photosynthetic bacteria can also utilize energy from sunlight, while certain types of bacteria can utilize energy from chemical reactions not involving respiration. The building blocks and energy are used to synthesize more cells for growth and also for reproduction.

In the treatment of wastewater three types of overall processes are distinguished to represent the conversion of organic wastes by micro-organisms. The classification is based on whether the environment where the process takes place is aerobic, anaerobic or photosynthetic. Under aerobic conditions (in the presence of oxygen), micro-organisms utilize oxygen to oxidize organic substances to obtain energy for maintenance, mobility and the synthesis of cellular material. Under anaerobic conditions (in the absence of oxygen) the micro-organisms utilize nitrates, sulphates and other hydrogen acceptors to obtain energy for the synthesis of cellular material from organic substances. Photosynthetic organisms use carbon dioxide as a carbon source, inorganic nutrients as sources of phosphate and nitrogen and utilize light energy to drive the conversion process.

Micro-organisms also produce waste products, some of which are desirable and some undesirable. Gases such as carbon dioxide and nitrogen are desirable, since they can be easily separated and do not produce pollution. Gases such as hydrogen sulphide and mercaptans, although easily separated require treatment for odour. Micro-organisms' cellular materials are organic in nature and can also cause pollution. It would be desirable if the cellular materials have undergone self oxidation (endogenous respiration utilizing own body cells) to produce non-biodegradable materials that are relatively stable. Self-oxidation is achieved when there is no substrate/food available.

The microbiological conversion reactions of organic waste into cellular material can be empirically represented as shown below.

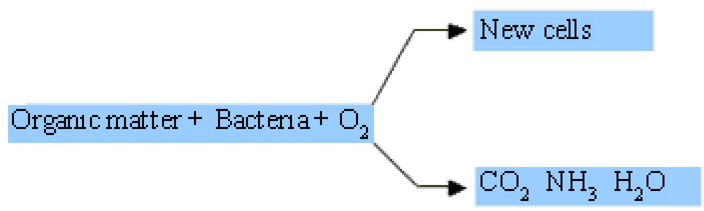
(i) Conversion under aerobic conditions (see diagram below):

Under aerobic conditions ammonia is further oxidized to nitrate. Phosphorus and sulphur contained in the organic substances are oxidized to phosphate and sulphate. These can be further utilized by the micro-organisms for synthesis.

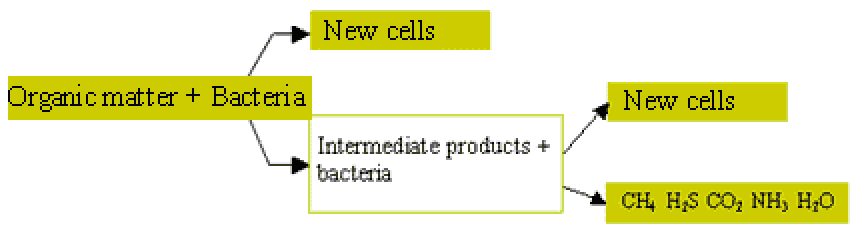
(ii) Conversion under anaerobic conditions (see diagram below):

Methane (CH_4) is a useful gaseous by-product of anaerobic conversion, because it can be combusted to produce heat/energy. On the other hand if it is released to the atmosphere without being combusted, it contributes to the greenhouse gas effect.

Aerobic Oxidation



Anaerobic Oxidation



(iii) Conversion under photosynthetic conditions:

$$aCO_2 + rH_2O + tNH_3 \xrightarrow{\text{Sunlight}} C_w H_x O_y N_z + bO_2$$

As shown by the conversion reactions (the utilization of organic wastes for food by micro-organisms) the product is mainly the cellular material of the micro-organisms i.e. more organisms are produced. The growth yield is the weight of micro-organisms produced per unit weight of organic substances consumed by the micro-organisms. The growth yield depends on the type of substrate and environmental conditions. The smaller the value of the growth yield the better it is for waste treatment, because less sludge is produced which requires disposal. Its value is usually between 0.2 and 0.5 for aerobic conversion, while the corresponding value for anaerobic conversion is smaller.

4.2 Microbial Growth Kinetics

Prokaryotic organisms such as bacteria reproduce mainly by binary fission (i.e., each cell gives two daughter cells). Growth of a microbial population is defined as an increase in numbers or an increase in microbial mass. Growth rate is the increase in microbial cell numbers or mass per unit time. The time required for a microbial population to double in numbers is the generation time or doubling time, which may vary from minutes to days.

Microbial populations can grow as batch cultures (closed systems) or as continuous cultures (open systems) (Marison, 1988a).

1. Batch Cultures

When a suitable medium is inoculated with cells, the growth of the microbial population follows the growth curve displayed in Figure 4-2, which shows four distinct phases.

i. Lag Phase

The lag phase is a period of cell adjustment to the new environment. Cells are involved in the synthesis of bio-chemicals and undergo enlargement. The duration of the lag phase depends on the cells prior history (age, prior exposure to damaging physical or chemical agents, culture medium).

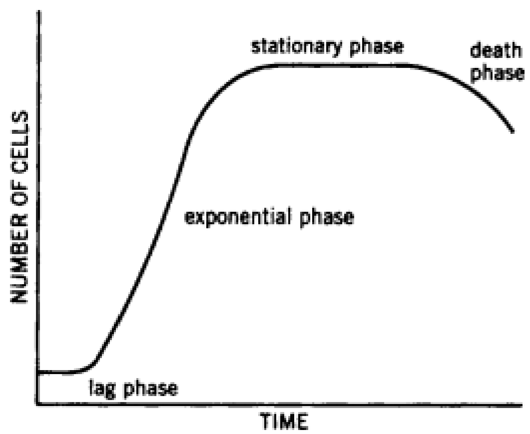


Figure 4-2 Microbial growth curve

For example, no lag phase is observed when an exponentially growing culture is transferred to a similar medium with similar growth conditions. Conversely, a lag period is observed when damaged cells are introduced into the culture medium.

ii. Exponential Growth Phase (Log Phase)

The number of cells increases exponentially during the log phase. The exponential growth varies with the type of microorganism and growth conditions (e.g., temperature, medium composition). Under favorable conditions, the number of bacterial cells (e.g., Escherichia coli) double every 15 - 20 min. The growth follows a geometric progression $(2^0 \rightarrow 2^1 \rightarrow 2^2 \rightarrow 2^n)$

$$X_t = X_o e^{\mu t} \qquad 4.1$$

Where μ - specific growth rate (h^{-1})

X_t - cell biomass or numbers after time t, and

X_o - initial number or biomass of cells

Using the natural logarithms on both sides of Eq. (4.1), we obtain

$$\ln X_t = \ln X_o + \mu t \qquad 4.2$$

Where μ is given by

$$\mu = \frac{\ln X_t - \ln X_o}{t} \qquad 4.3$$

Cells in the exponential growth phase are more sensitive to physical and chemical agents than those in the stationary phase.

iii. Stationary Phase

The cell population reaches the stationary phase because microorganisms cannot grow indefinitely, mainly because of lack of nutrients and electron acceptors, and the production and the accumulation of toxic metabolites. Secondary metabolites (e.g., certain enzymes, antibiotics) are produced during the stationary phase. There is no net growth (cell growth is balanced by cell death or lysis) of the population during the stationary phase.

iv. Death Phase

During this phase, the death (decay) rate of the microbial population is higher than the growth rate. Cell death may be accompanied by cell lysis. The viable count of microorganisms decreases, although the turbidity of the microbial suspension may remain constant.

2. Continuous Culture of Microorganisms

So far, we have described the growth kinetics of batch cultures. Maintenance of microbial cultures at the exponential growth phase over a long period of time can be achieved by growing continuously the cells in a completely mixed reactor in which a constant volume is maintained. The most commonly used device is the chemostat (Figure 4-3), which is essentially a complete-mix bioreactor without recycle. In addition to the flow rate of growth-limiting substrate, environmental parameters such as oxygen level, temperature, and pH are also controlled. The substrate is added continuously at a flow rate Q to a reactor with a volume V containing concentration X of microorganisms. The dilution rate D, the reciprocal of the hydraulic retention time t, is given by:

$$D = \frac{Q}{V} = \frac{1}{t} \qquad 4.4$$

Where D - dilution rate ($time^{-1}$)

V - reactor volume (L)

Q - flow rate of substrate S (L/time), and

t - time

In continuous flow reactors, microbial growth is described by

$$\frac{dX}{dt} = \mu X - DX = X(\mu - D) \qquad 4.5$$

$$X_t = X_o e^{(\mu - D)t} \qquad 4.6$$

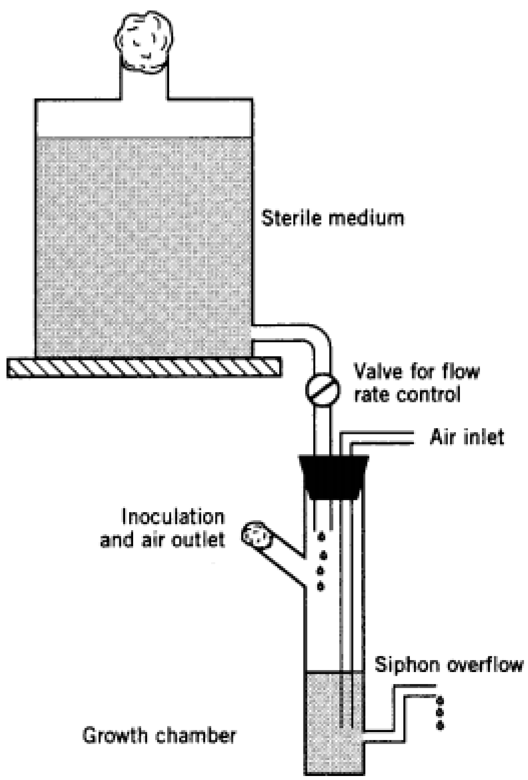


Figure 4-3 Chemostat for continuous culture of microorganisms

During chemostat cultivation, equilibrium is established (steady state) at which the growth rate of the cells equals the dilution rate. The higher the dilution rate, the faster the organisms are allowed to grow. Above a given dilution rate the cells will not be able to grow any faster and the culture will be washed out of the reactor. The chemostat thus offers the opportunity to study the properties of organisms at selected growth rates.

Equation (2.26) and (2.27) shows that the supply rate of the limiting substrate controls the specific growth rate, μ. At $D > \mu_{max}$, we observe a decrease in cell concentration and a washout of the population. Cell washout starts at the critical dilution rate D_c, which is approximately equal to μ_{max}.

3. Other Kinetic Parameters

There are three important parameters in microbial growth kinetics: growth yield Y, specific growth rate μ, and specific substrate uptake rate q.

A more simplified equation showing the relationship between the three parameters is the following:

$$\mu = Yq \qquad 4.7$$

Where μ - specific growth rate ($time^{-1}$),

Y - growth yield (mg cells formed per mg of substrate removed), and

q – specific substrate uptake rate (mg/L/day).

i. Growth Yield

The rate of increase of microorganisms in a culture ($\frac{dX}{dt}$) is proportional to the rate of substrate uptake/removal ($\frac{dS}{dt}$) by microbial cells:

$$\frac{dX}{dt} = Y\frac{dS}{dt} \qquad 4.8$$

Where Y - growth yield coefficient expressed as mg cells formed per mg of substrate used,

dX/dt -rate of increase in microorganism concentration (mg/L/day), and

dS/dt – rate of substrate removal (mg/L/day).

It reflects the efficiency of conversion of substrate to cell material. The yield coefficient Y is obtained as

$$Y = \frac{X - X_o}{S_o - S} \qquad 4.9$$

Where S_o and S - initial and final substrate concentrations, respectively (mg/L or mol/L),

X_o and X - initial and final microbial concentrations, respectively

Several factors influence the growth yield: type of microorganisms, growth medium, substrate concentration, terminal electron acceptor, pH, and incubation temperature.

Yield coefficients for several bacterial species are within the range 0.4 – 0.6 (Heijnen and Roels, 1981).

For a pure microbial culture growing on a single substrate, the growth yield Y is assumed to be constant. However, in the environment, particularly in wastewater, there is a wide range of microorganisms, few of which are in the logarithmic phase. Many are in the stationary or in the declining phase of growth. Some of the energy will be used for cell maintenance. Thus, the growth yield Y must be corrected for the amount of cell decay occurring during the declining phase of growth. This correction will give the true growth yield coefficient, which is lower than the measured yield. Equation (4.7) becomes:

$$\mu = Yq - k_d \qquad 4.10$$

Where k_d is the endogenous decay coefficient (day^{-1})

ii. Specific Substrate Uptake Rate q

The specific substrate uptake (removal) is given by

$$q = \frac{dS}{dt}/X \qquad 4.11$$

Where q ($time^{-1}$) is given by the Monod's equation:

$$q = q_{max}\frac{[S]}{K_s + [S]} \qquad 4.12$$

iii. Specific Growth Rate μ

This is given by:

$$\mu = \frac{dX/dt}{X} \qquad 4.13$$

Where μ (day^{-1}) is given by Monod's equation:

$$\mu = \mu_{max}\frac{[S]}{K_s + [S]} \qquad 4.14$$

The in situ specific growth rate of bacteria in wastewater was measured using the labeled thymidine growth assay (thymidine is a precursor of DNA in cells). In an aerobic tank, the specific growth rate μ was 0.5 d^{-1} (doubling time t_d = 1.4d) whereas in an anaerobic tank μ was equal to 0.2 d^{-1} (t_d = 3.9d) (Pollard and Greenfield, 1997). In waste treatment, the reciprocal of μ is the biological solid retention time θ_c, that is

$$\mu = \frac{1}{\theta_c} \qquad 4.15$$

Thus

$$\frac{1}{\theta_c} = Yq - k_d \qquad 4.16$$

4. Physical and Chemical Factors Affecting Microbial Growth

i. Substrate Concentration.

The relationship between the specific growth rate μ and substrate concentration S is given by the Monod's equation (Fig. 2.14a):

$$\mu = \mu_{max}\frac{[S]}{K_s + [S]} \qquad 4.17$$

Where μ_{max} - maximum specific growth rate (h^{-1})

S - substrate concentration (mg/L),

K_s - half-saturation constant (mg/L).

K_s is the substrate concentration at which the specific growth rate is equal to $\mu_{max}/2$. K_s represents the affinity of the microorganism for the substrate. μ_{max} and K_s are influenced by temperature, type of carbon source, and other factors.

Monod's equation can be linearized using the Lineweaver–Burke equation:

$$\frac{1}{\mu} = \frac{K_s}{\mu_{max}[S]} + \frac{1}{\mu_{max}} \qquad 4.18$$

Figure 4-4 shows a plot of $1/\mu$ vs $1/S$. The slope, y-intercept, and x-intercept are (K_s/μ_{max}), ($1/\mu_{max}$), and ($-1/K_s$), respectively. This plot allows the computation of K_s and μ_{max}. K_s values for individual chemicals found in wastewater are between 0.1 and 1.0 mg/L (Hanel, 1988).

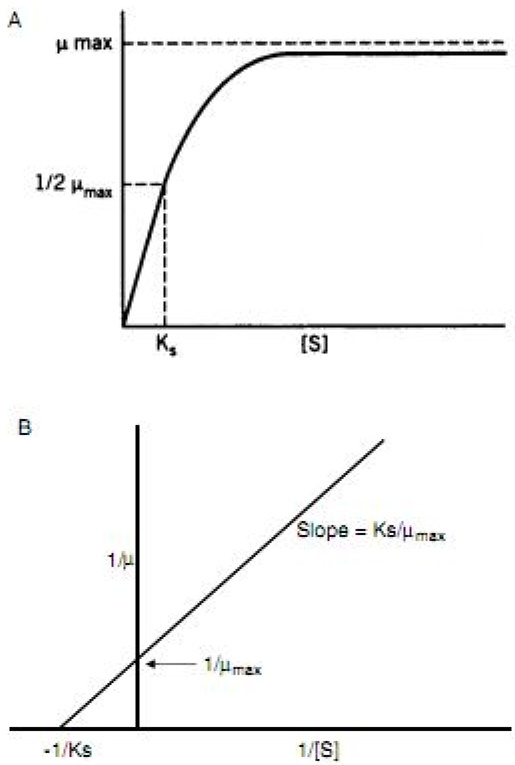


Figure 4-4 Relationship between the specific growth rate μ and substrate concentration S

(A) Monod's saturation curve; (B) Lineweaver–Burke plot

ii. Temperature

This is one of the most important factors affecting microbial growth and survival. Microbial growth can occur at temperatures varying from below freezing to more than 100°c. Based on the optimum temperature for growth, microorganisms are classified as mesophiles, psychrophiles, thermophiles, or extreme thermophiles.

Microbial growth rate is related to temperature by the Arrhenius equation:

$$\mu = Ae^{-E/RT} \qquad 4.19$$

Where A - constant,

E - activation energy (kcal/mole),

R - gas constant, and T - absolute temperature (K)

Psychrophiles can grow at low temperatures because their cell membrane has a high content of unsaturated fatty acids, which helps maintain membrane fluidity, whereas a high content of saturated fatty acids help thermophiles function at high temperatures. The decreased m at high temperatures is due to the thermal denaturation of proteins, particularly enzymes, as well as changes in membrane structure, leading to alterations in cell permeability.

iii. pH

Biological treatment of wastewater occurs generally at neutral pH. In general, the optimum pH for bacterial growth is around 7, although some may be obligately acidophilic (e.g., Thiobacillus, Sulfolobus) and thrive at pH < 2. Fungi prefer acidic environments with a pH of 5 or lower. Cyanobacteria grow optimally at pH higher than 7. Bacterial growth generally results in a decrease of the pH of the medium through the release of acidic metabolites (e.g., organic acids, H_2SO_4). Conversely, some microorganisms can increase the pH value of their surrounding milieu (e.g., denitrifying bacteria, algae).
pH affects the activity of microbial enzymes. It affects the ionization of chemicals and thus plays a role in the transport of nutrients and toxic chemicals into the cell.

iv. Oxygen Level

Microorganisms can grow in the presence or in the absence of oxygen. There are divided into strict aerobes, facultative anaerobes (can grow in the presence or in the absence of oxygen), and strict anaerobes. Aerobic microorganisms use oxygen as the terminal electron acceptor in respiration. Anaerobic counterparts use other electron acceptors such as sulfate, nitrate, or CO_2. Some microorganisms are micro-aerophilic and require low levels of oxygen for growth. Through their metabolism, aerobes may render the environment suitable for anaerobes by using oxygen.
Upon reduction, oxygen forms toxic products such as superoxide (O_2^-), hydrogen peroxide (H_2O_2), or hydroxyl radicals. However, microorganisms have acquired enzymes to deactivate them. For example, H_2O_2 is destroyed by catalase and peroxidase enzymes, whereas O_2^- is deactivated by superoxide dismutase. Catalase and superoxide dismutase-catalyzed reactions are represented by:

$$2\,O_2^- + 2\,H^+ \xrightarrow[\text{superoxide dismutase}]{} O_2 + H_2O_2$$

$$2\,H_2O_2 \xrightarrow[\text{catalase}]{} 2\,H_2O + O_2$$

4.3 Biological Wastewater Treatment

Purpose:

The idea behind all biological methods of wastewater treatment is to introduce contact with bacteria (cells) which feed on the organic materials in the wastewater, thereby reducing its BOD content. In other words, the purpose of biological treatment is BOD reduction.

Typically, wastewater enters the treatment plant with a BOD higher than 200mg/L, but primary settling has already reduced it to a certain extent (30 – 35% of the original) by the time it enters the biological component of the system. It needs to exit with a BOD content no higher than about 20 - 30mg/L, so that after dilution in the nearby receiving water body (river, lake), the BOD is less than 2 - 3mg/L.

Principle:

Simple bacteria (cells) eat the organic material present in the wastewater. Through their metabolism, the organic material is transformed into cellular mass, which is no longer in solution but can be precipitated at the bottom of a settling tank or retained as slime on solid surfaces or vegetation in the system. The wastewater exiting the system is then much clearer than it entered.

A key factor is the operation of any biological system is an adequate supply of oxygen.

Indeed, cells need not only organic material as food but also oxygen to breath, just like humans. Without an adequate supply of oxygen, the biological degradation of the waste is slowed down, thereby requiring a longer residency time of the wastewater in the system. For a given flow rate of wastewater to be treated, this translates into a system with a larger volume and thus taking more space.

4.4 Types of Biological Process for Wastewater Treatment

The common methods of biological wastewater treatment are:

a) Aerobic processes such as trickling filters, rotating biological contactors, activated sludge process, oxidation ponds and lagoons, oxidation ditches,
b) Anaerobic processes such as anaerobic digestion, and
c) Anoxic processes such as denitrification.

The major biological wastewater treatment processes are shown in Table 4-1.

Table 4-1 Major biological treatment processes used for wastewater treatment

Type	Common name	Use
Aerobic Processes		
Suspended growth	Activated Sludge Process	
	- Conventional (Plug flow)	Carbonaceous BOD
	- Step aeration, Modified aeration	“
	- Contact Stabilization	“
	- Extended Aeration, Oxidation Ditch	“ + Nitrification
	Aerated Lagoons	Carbonaceous BOD
	Aerobic Digestion High	
	Rate Algal Ponds	Carbonaceous BOD
Attached growth	Trickling Filters	
	- Low rate	Carbonaceous BOD
	- High rate	“
	Rotating Biological Contactors (RBC)	“
Anaerobic Processes		
Suspended growth	Anaerobic Digestion	
	- Standard rate	Stabilization
	- High rate Single	“
Attached growth	Anaerobic Contact Proc.	Carbonaceous BOD
	Anaerobic Filter Process	“
	Anaerobic Lagoons	“
Anoxic Processes		
	- Suspended growth	Denitrification
	- Fixed film	“

4.4.1 Trickling Filters

The conventional trickling filters and their improved forms, known as high rate trickling filters are now almost universally adopted for giving secondary treatment to sewage. These filters, also called as percolating filters or sprinkling filters, consist of tanks of coarser filtering media, over which the sewage is allowed to sprinkle or trickle down, by means of spray nozzles or rotary distributors. The percolating sewage is collected at the bottom of the tank through a well designed under-drainage system. The purification of the sewage is brought about mainly by the aerobic bacteria, which form a bacterial film around the particles of the filtering media. The action due to the mechanical straining of the filter bed is much less. In order to ensure the large scale growth of the aerobic bacteria, sufficient quantity of oxygen is supplied by providing suitable ventilation facilities in the body of the filter; and also to some extent by the intermittent functioning of the filter.

The effluent obtained from the filter must be taken to the secondary sedimentation tank for settling out the organic matter oxidized while passing down the filter. The sewage influent entering the filter must be given pre-treatments including screening and primary sedimentation.

Construction and Operation of Trickling Filters

Trickling filter tanks are generally constructed above the ground. They may either be rectangular or more generally circular (Figure 4-5and Figure 4-6). Rectangular filters are provided with a network of pipes having fixed nozzles which spray the incoming sewage in to the air which then falls over the bed of the filter, under gravity.

The circular filter tanks on the other hand, are provided with rotary distributors having a number of distributing arms (generally four arms are used). These distributors rotate around a central support either by an electric motor, or more generally by the force of reaction on the sprays. Such self-propelled reaction type of distributors (Figure 4-7) is now-a-days preferred and used. The rate of revolutions varies from 2RPM for small distributors to less than $\frac{1}{2}$ RPM for large distributors. The advantage of having two or more arms is not only to get reaction sufficient to rotate the entire mechanism but is also to pass the fluctuating demands by taking low flows in two arms, and the remaining two arms coming into operation only at the times of higher flows. The distributing arms should remain about 15 to 20cm above the top surface of the filtering media in the tank.

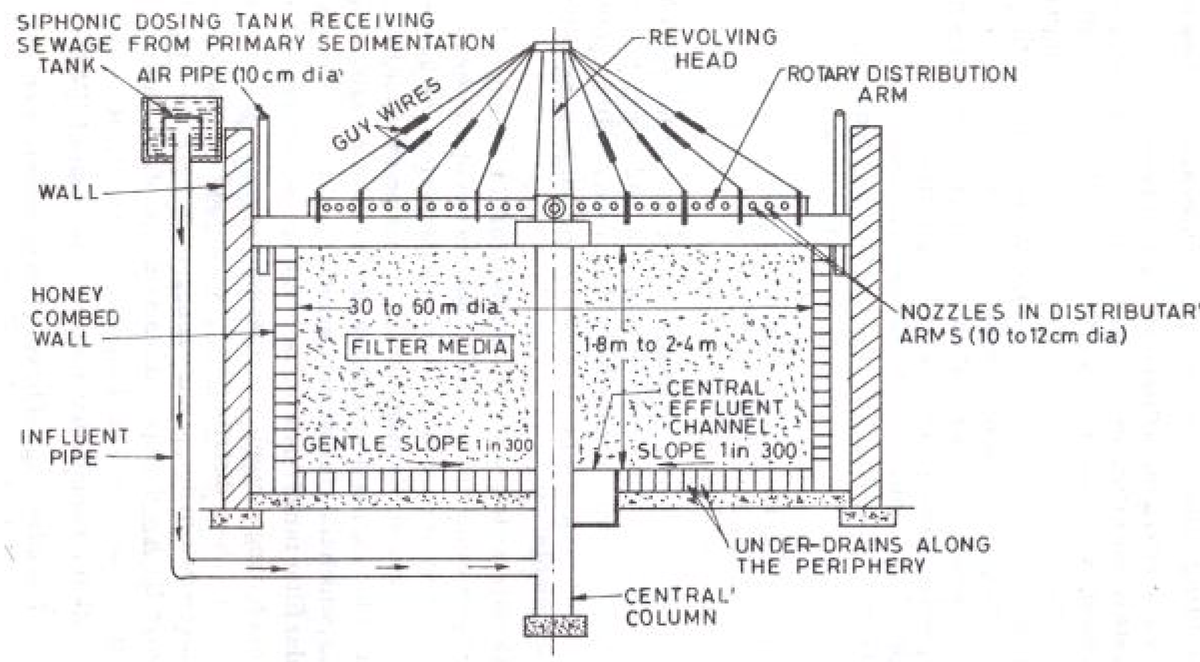


Figure 4-5 Typical section of a conventional circular trickling filter

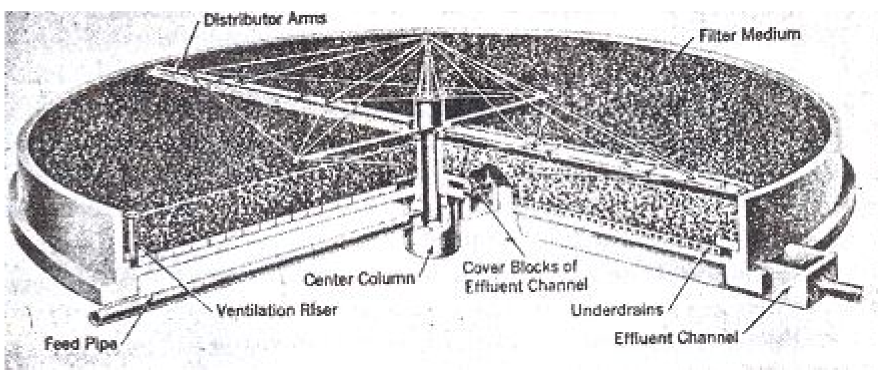


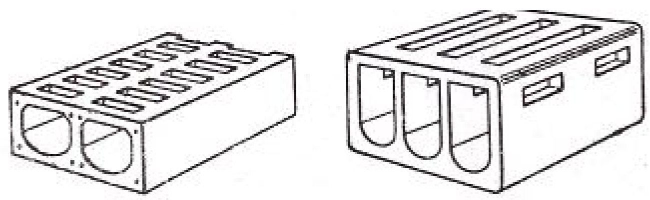
Figure 4-6 Photographic view of a conventional circular trickling filter with rotary distributors

Figure 4-7 Photographic view of rotary distributors

There is an important difference between the action of rotary distributors and that of spray nozzles. With a rotary distributor, the application of sewage to the filter is practically continuous; whereas with spray nozzles, the filter is dosed for 3 to 5 minutes, and then rested for 5 to 10 minutes before the next application. In any case, however, dosing tanks with siphons, receiving sewage from the primary clarifiers and supplying it to the filters at regular intervals, are used. The dosing tank for a filter with circular distributors will, however, be designed to have a smaller capacity (i.e. about 1 to 3 minutes detention capacity), as against a higher capacity (of about 5 to 10 minutes detention capacity) for filters with spray nozzles.

The filtering media, as pointed out earlier, consists of coarser materials like cubically broken stones or slag, free from dust and small pieces. The size of the material used may vary between 25 to 75mm. The filtering material should be washed before it is placed in position. The quality of the stone used should be such as not to be easily affected by acidic sewage, and should be sufficiently hard. Its resistance to freezing and thawing is another important property, especially for northern regions. Usually, stones from rocks like granite or limestone may be used.

The depth of the filtering media may vary between 2 to 3 meters. The filtering material may be placed in layers; with coarsest stone used near the bottom, and. finer material towards the top.
The walls of the filter tank are made honey-combed or otherwise provided with openings for circulation of air, all through: Sometimes, instead of constructing the supporting walls, the filtering material may be stacked above the ground with its natural angle of repose, so as to ensure better circulation of air from the surrounding atmosphere.
Sometimes, forced ventilation, by forcing the air vertically upwards through the filter by the use of fans or other mechanical equipment, may be used; but it has not been found to increase the capacity of well-constructed trickling filters.
A satisfactory ventilation is achieved when properly designed under drains having adequate openings are provided under the filter bed. Besides ensuring satisfactory drainage, such drains, will also ensure satisfactory ventilation and aeration of the filter bed. Vitrified clay blocks (Figure 4-8) are generally used as under-drains. These blocks have top openings of such size that the stone can be placed directly on them, and yet they furnish flow channels -with' sufficient capacity for the heaviest hydraulic loading. These blocks are laid on a reinforced concrete floor, (about 10 to 15cm thick) which is sloped gently (at about 1 in 300) towards the main effluent rectangular channel.



(a) Under-drain block for standard trickling filters.

(b) Under-drain block for high rate trickling filters with heavy hydraulic loading

Figure 4-8 Under drains

This main effluent channel may be provided adjoining the central column of the distributor as shown in Figure 4-8 and Figure 4-8; or may be provided along the circular periphery of the filter. The flow in this channel has characteristics similar to the flow in a wash-water trough of a rapid sand filter used in water supplies. The slope of the channel should be sufficient to ensure a flow velocity of about 0.9 m/sec. The depth and the width of this central channel should be such that the maximum flow is carried below the level of the under-drains.

Types of Trickling Filters

Trickling filters can be broadly classified into:

(1) Conventional trickling filters or standard rate or low rate trickling filters
(2) High rate filters or High rate trickling filters

Strictly speaking, all what we have spoken so far is about conventional or standard rate trickling filters. The high rate filters of modern advancements, also function on the same lines, and are having the same constructional details, but with the difference that provision is made in them for recirculation of sewage through the filter, by pumping a part of the filter-effluent to the primary settling tank, and re-passing through it and the filter. The high rate filters make it possible to pass sewage at greater loadings, thus requiring lesser space and lesser filter media. The process of 'recirculation' and its use in making high rate filters shall be discussed thoroughly later.

Design of Trickling Filters

The design of the trickling filter primarily involves the design of the diameter of the circular filter tank and its depth. The design of the rotary distributors and under-drainage system is also involved in the filter design.

The design of the filter size is based upon the values of the filter-loadings adopted for the design. This loading on a filter can be expressed in two ways:

(i) By the quantity of sewage applied per unit of surface area of the filter per day

This is called hydraulic-loading rate and expressed in million liters per hectare per day. The value of hydraulic loading for conventional filters may vary between 22 and 44 (normally 28) million liters per hectare per day. The hydraulic loading can still be increased to about 110 to 330 (normally 220) M.L/ha/day in the high rate trickling filters.

(ii) By the mass of BOD per unit volume of the filtering media per day

This is called organic loading rate, and expressed in kg of BOD_5 per hectare meter of the filter media per day. The value of organic loading for conventional filters may vary between 900 to 2200 kg of BOD_5 per ha-m. This organic loading value can be further increased to about 6000 - 18000 kg of BOD_5 per ha-m in high rate trickling filters.

With an assumed value of organic loading (as between 900 to 2200kg/ha-m), we can find out the total volume of the required filter, by dividing the total BOD_5 of the sewage entering the filter per day in kg by the assumed value of the organic loading. The organic loading can thus, decide the volume of the filter.

The hydraulic loading, on the other hand, gives us the area of the filter required; when the total sewage volume entering the filter per day is divided by the hydraulic loading, (assumed between 22 and 44 Ml/ha/day).

Knowing the volume and area of the cylindrical filter, we can easily find out its diameter and depth.

It may also be mentioned here that the filter diameter and depth is designed for average value of sewage flow. The rotary distributors, under-drainage system, and other connected pipe lines etc. are, however, designed for peak flow and of course checked for the average flow. Moreover, since the rotary distributors are available indigenously only up to 60m in length, it is desirable to keep the diameter of the filter tank up to a maximum of 60m. If the required filter diameter is more than 60m, then it is better to use more units of lesser diameter.

Performance of Conventional Filters and Their efficiencies

The effluent obtained from a conventional trickling filter plant is highly nitrified and stabilized. The BOD is reduced to about 80 to 90% of the original value. The BOD left in the effluent is generally less than 20ppm or so. The sludge obtained in the secondary clarifier is thick, with moisture content of about 92%. It is heavy and easily digestible. The filter is very flexible, and can even take intermittent shock loads without any detrimental effects. Hence, a conventional or standard rate filter plant is very useful to medium towns and industrial cities requiring full treatment of sewage.

The efficiency of such a conventional filter plant can be expressed by the equation evolved by National Research Council of U.B.A., and given by:

$$\eta(\%) = \frac{100}{1 + 0.0044\sqrt{u}} \qquad 4.20$$

Where, η = Efficiency of the filter and its secondary clarifier, in terms of percentage of applied BOD removed

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

This equation shows that the BOD removed by the filter plant depends upon the organic loading adopted. Greater is the loading, lesser is the efficiency and thus lesser BOD is removed. This equation holds good when there is no recirculation.

Recirculation of Treated Sewage

Recirculation of sewage is an essential and important feature of high rate filters. The recirculation consists in returning a portion of the treated or partly treated sewage to the treatment process. Usually the return is from the secondary settling tank to the primary settling tank or to the dosing tank of the filter as shown in Figure 4-9. Sometimes, the effluent from the filter itself before it enters the secondary clarifier may be sent back to the primary clarifier.

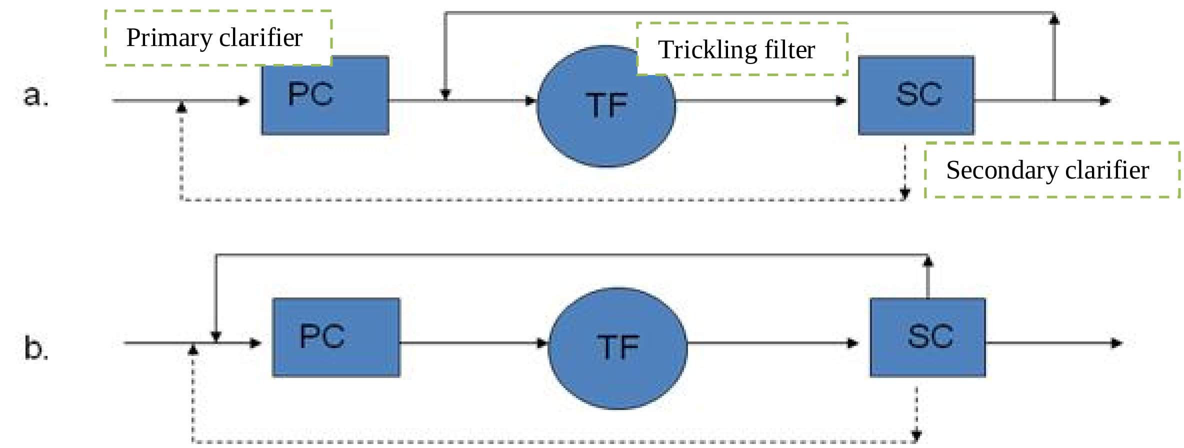


Figure 4-9 Single Stage commonly adopted Recirculation Process

In some other cases, and to obtain better efficiency, two stage recirculation processes may be adopted. A two stage recirculation process consists of having two filters arranged in series, as shown in Figure 4-10. Various other combinations are possible.

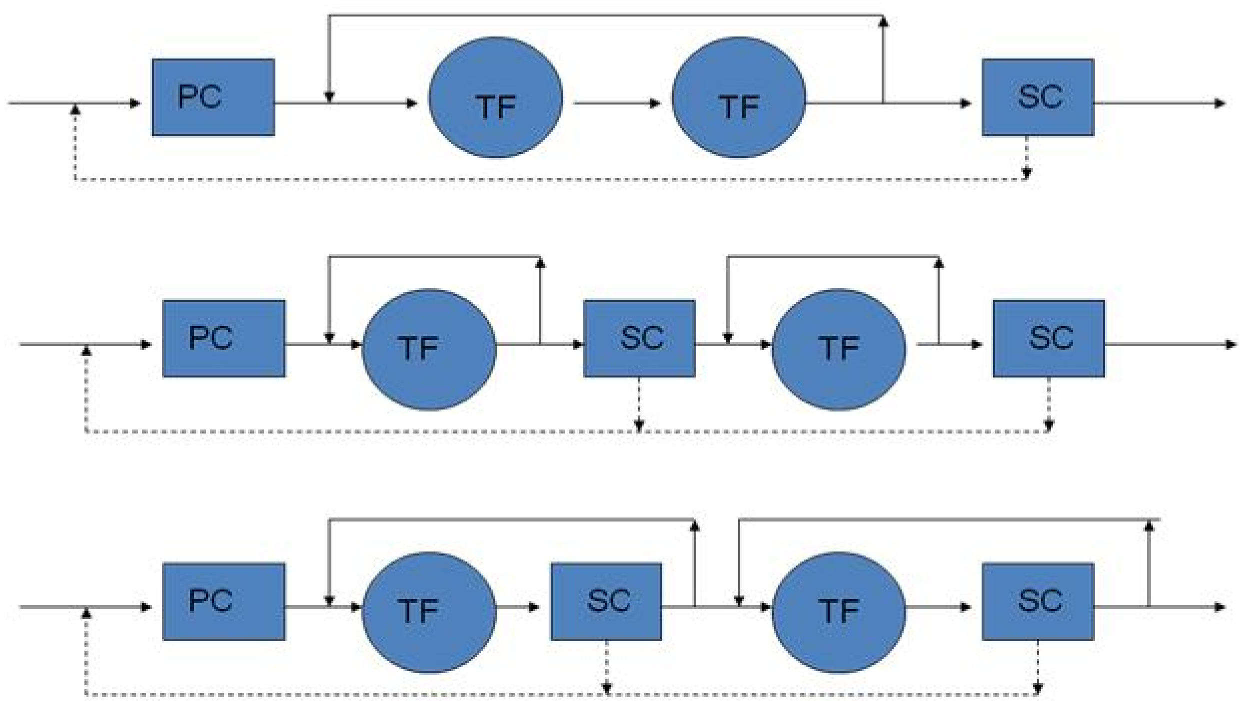


Figure 4-10 Two stage commonly adopted recirculation process

Recirculation improves the operating results of filters, because of the following reasons:

(i) Recirculation allows continuous dosing of the filters, irrespective of the fluctuations in flow.

(ii) Recirculation equalizes and reduces loading, thereby increasing the efficiency of the filter.

(iii) Recirculation provides longer contact of the applied sewage with the bacterial film on the contact media, thereby seeding it with bacteria, and accelerating the biological oxidation process.

(iv) The influent remains fresh all the time and also helps in reducing odors. The fly nuisance is also comparatively less.

It may, however, be noted that although the effluent would greatly be reduced in its BOD content because of the re-circulated flow, yet a large volume of sewage through the filter tends to wash off the filter before nitrification has had time to take place, resulting in loss of nitrates in the effluent, thereby slightly lowering the quality of the effluent. For this reason, a high rate filter plant with single stage recirculation may not show as good results as those obtained from a conventional trickling filter plant. For comparable or better results, two high rate filters are constructed and operated in series, as shown in Figure 4-10 As pointed out earlier, various recirculation schemes may be used, the most common being to pump back apart of the effluent from each filter to the influent of the same filter.

Efficiency of High Rate Filters

The efficiency of high rate filters depend upon the volume of the re-circulated flow (in comparison to the volume of raw sewage) and also upon the organic loading.

The ratio $(\frac{R}{I})$ of the volume of sewage re-circulated (R) to the volume of raw sewage (I) is called recirculation ratio, and is an important feature in obtaining the efficiency of the filter plant (or to work out the required degree of treatment for obtaining certain efficiency), The recirculation ratio is connected to another term called ***recirculation factor*** (F) by the relation:

$$F = \frac{1 + \frac{R}{I}}{\left(1 + 0.1 * \frac{R}{I}\right)^2} \qquad 4.21$$

The recirculation factor (F) also represents the number of effective passages through the filter. Thus, when there is no recirculation and $\frac{R}{I}$ is zero, F is unity.

The efficiency of the single stage high rate trickling filter can then be worked out by using the equation,

$$\eta(\%) = \frac{100}{1 + 0.0044\sqrt{\frac{Y}{V * F}}} \qquad 4.22$$

Where, Y - the total organic loading in kg/day applied to the filter, i.e. the total BOD in kg. The term $\frac{Y}{V*F}$ is also called unit organic loading on filter, i.e., u

V - Filter volume in hectare meters

F - Recirculation factor

In a two stage filter, the efficiency achieved in the first stage will be obtained by the previous equation; and in the second stage, it is obtained as:

Final efficiency in the two stage filter

$$\eta' = \frac{100}{1 + \frac{0.0044}{1-\eta} * \sqrt{\frac{Y'}{V' * F'}}} \qquad 4.23$$

Where, Y' - Total BOD in effluent from first stage in kg/day

V' - Volume of second stage filter in ha-m

F' - Recirculation factor for the second stage filter

η'- Final efficiency obtained after two stage filtration

These equations are very important, as they form the basis of designing high rate filters.

Example 4-1

a. Design suitable dimensions of circular trickling filter units for treating 5 million liters of sewage per day. The BOD of sewage is 150mg/l.
b. Also design suitable dimensions for its rotary distribution system, as well as the under-drainage system.

Solution

Total BOD present in sewage to be treated per day

$$= 5 * 150\text{kg} = 750\text{kg}$$

Assuming the value of organic loading is 1500kg/ha.m/day [i.e. between 900 to 2200kg/ha.m/day]

The volume of filtering-media required

$$= \frac{750}{1500}\,\text{ha.m} = 0.5\text{ha.m} = 5000\text{m}^3$$

Assuming the effective depth of filter as 2m,

The surface area of the filter required

$$= \frac{5000}{2} = 2500\text{m}^2$$

Using a circular trickling filter of diameter 40m,

The number of units required

$$= \frac{\text{Total area required}}{\text{Area of one unit}} = \frac{2500}{\frac{\pi}{4} * 40^2} \approx 2\text{ units}$$

Check for Hydraulic loading

The surface area of the filter bed required can also be worked out by assuming the value of hydraulic loading, say as, 25 million liters per hectare per day (between 22 to 44ML/ha/day)

Surface area required

$$= \frac{\text{Total sewage to be treated per day}}{\text{Hydraulic loading per day}}$$

$$= \frac{5}{25} * 10{,}000 = 2000\text{m}^2$$

The surface area chosen is 2500m^2, which is greater than 2000 m^2, and hence safe.

Hence, 2 units each of 40m diameter and 2m effective depth (i.e. 2.6m overall depth), can be adopted. An extra third unit as stand-by may also be constructed.

Example 4-2

The sewage is flowing @ 4.5Million liters per day from a primary clarifier to a standard rate trickling filter. The 5-day BOD of the influent is 160mg/l. The value of the adopted organic loading is to be 160 gm/m^3/day, and surface loading 2000 l/m^2/day. Determine the volume of the filter and its depth. Also calculate the efficiency of this filter unit.

Solution

Total 5-day BOD present in sewage

$$= \frac{160 * 4.5 * 10^6}{10^3} = 720{,}000\text{gm/day}$$

Volume of the filter media required

$$= \frac{\text{Total BOD}}{\text{Organic loading}} = \frac{720{,}000\text{gm/day}}{160\text{gm/m}^3.\text{day}} = 4{,}500\text{m}^3$$

Surface area required for the filter

$$= \frac{\text{Total flow}}{\text{Hydraulic loading}} = \frac{4.5 * 10^6\ \text{l/d}}{2000\ \text{l/m}^2.\text{d}} = 2250\text{m}^2$$

Depth of the bed required

$$= \frac{4500}{2250} = 2\text{m}$$

Efficiency of the filter is given by Eq. (4.20) as:

$$\eta(\%) = \frac{100}{1 + 0.0044\sqrt{u}}$$

Where, u = organic loading in kg/ha-m/day

Organic loading, u = 160 gm/m^3/day

$$1\ \text{hectare.m} = 10^4\ \text{m}^2.\text{m} = 10^4\ \text{m}^3$$

$$u = 1600\text{kg/ha.m/day}$$

Hence,

$$\eta(\%) = \frac{100}{1 + 0.0044\sqrt{1600}} = 85.03\%$$

Comparison of Conventional and High rate Trickling Filters

Table 4-2 Conventional vs high rate trickling filters

S. No	Characteristics	Conventional filters	High rate filters
1	Depth of filter media	Varies between 1.6 to 2.4m	Varies between 1.2 to 1.8m
2	Size of filter media	25 to 75	25 to 60
3	Land required	More land area is required as the filter loading is less.	Less land area is required as the filter loading is more.
4	Cost of operation	It is more for treating equal quantity of sewage	It is less for treating equal quantity of sewage.
5	Method of operation	Continuous application, less flexible requiring less skilled supervision.	Continuous application, more flexible, and more skillful operation is required.
6	Types of effluent produced	The effluent is highly nitrified and stabilized, with BOD in effluent $\leq$ 20ppm	The effluent is nitrified up to nitrite stage only and is thus less stable, and hence it is of slightly inferior quality. BOD in effluent $\geq$ 30ppm.
7	Dosing interval	It generally varies between 3 to 10 minutes. The sewage is generally not applied continuously but is applied at intervals.	It is not more than 15 seconds, and the sewage is thus applied continuously
8	Filter loading values (i) Hydraulic loading (ii) Organic loading	 Varies between 20 to 44ML per hectare per day Varies between 900 to 2200kg of BOD_5 per ha-m of filter media per day.	 Varies between 110 to 330ML per hectare per day Varies between 6000 to 18,000kg of BOD_5 per hectare meter of filter media per day
9	Recirculation system	Not provided generally	Always provided for increasing hydraulic loading
10	Quality of secondary sludge produced	Black, highly oxidized with slight fine particles	Brown, not fully oxidized with fine particles

4.4.2 Activated Sludge Process

The activated sludge process provides an excellent method of treating either raw sewage or more generally the settled sewage. The sewage effluent from primary sedimentation tank, which is, thus normally utilized in this process is mixed with 20 to 30 percent of own volume of activated sludge which contains a large concentration of highly active aerobic micro-organisms.

The microorganisms utilize the absorbed organic matter as a carbon and energy source for cell growth and convert it to cell tissue, water, and oxidized products (mainly carbon dioxide, CO_2). Some bacteria attack the original complex substance to produce simple compounds as their waste products. Other bacteria then use these waste products to produce simpler compounds until the food is used up.

The mixture of wastewater and activated sludge in the aeration basis is called mixed liquor. The biological mass (biomass) in the mixed liquor is called the mixed liquor suspended solids (MLSS) or mixed liquor volatile suspended solids (MLVSS). The MLSS consists mostly of microorganisms, non-biodegradable suspended organic matter, and other inert suspended matter. The microorganisms in MLSS are composed of 70 to 90 percent organic and 10 to 30 percent inorganic matter (Okun 1949, WEF and ASCE 1996a). The types of bacterial cell vary depending on the chemical characteristics of the influent wastewater tank conditions and the specific characteristics of the microorganisms in the flocs. Microbial growth in the mixed liquor is maintained in the declining or endogenous growth phase to insure good settling properties.

The mixture enters an aeration tank where the micro-organisms (coated around the sludge solids) and the sewage are intimately mixed together with a large quantity of air for about 4 to 8 hours. Under these conditions, the moving organisms will oxidize the organic matter and the suspended and colloidal matter tends to coagulate and form a precipitate which settles down readily in the secondary settling tank. The settled sludge (containing micro organisms) called activated sludge is then recycled to the head of the aeration tank to be mixed again with the sewage being treated.

New activated sludge is continuously being produced by this process and a portion of it being utilized and sent back to the aeration tank whereas the excess portion is disposed of properly along with the sludge collected during primary treatment after digestion.

The effluent obtained from a properly operated activated sludge plant is of high quality usually having a lower BOD than that of a trickling filter plant. BOD removal is up to 80 - 95 percent, and bacteria removal is up to 90 - 95 percent. Moreover, land area required is also quite less. But, however, in this process, a rather close degree of control is necessary in operation to ensure that

(i) an ample supply of oxygen is present
(ii) there is intimate and continuous mixing of the sewage and the activated sludge

(iii) the ratio of the volume of activated sludge added to the volume of sewage being treated is kept practically constant

Moreover, there is the problem of obtaining activated sludge at the start of a new plant. Hence, when a new plant is put into operation a period of about 4 weeks may be required to form a suitable return sludge and during this period almost all the sludge from the secondary sedimentation tank will be returned through the aeration tank. A new plant may also sometimes be seeded with the activated sludge from another plant so as to quickly start the process in the new plant.

Various Operations and Units of an Activated Sludge Plant

1. Aeration Tanks of an Activated Sludge Plant

A typical flow diagram for a conventional activated sludge plant is shown in Figure 4-12. As pointed out earlier, the removal of grit and larger solids by screening in grit chambers and primary sedimentation tanks is generally considered necessary before aeration. The pre-removal of these settleable solids is helpful in preventing deposits on aeration devices, and thereby not reducing their efficiencies. Moreover, such materials, if not pre-removed may settle down in the aeration tank and by decomposition interfere with the treatment process. Accordingly, grit removal, screening, and primary sedimentation are considered necessary for a conventional activated sludge process.

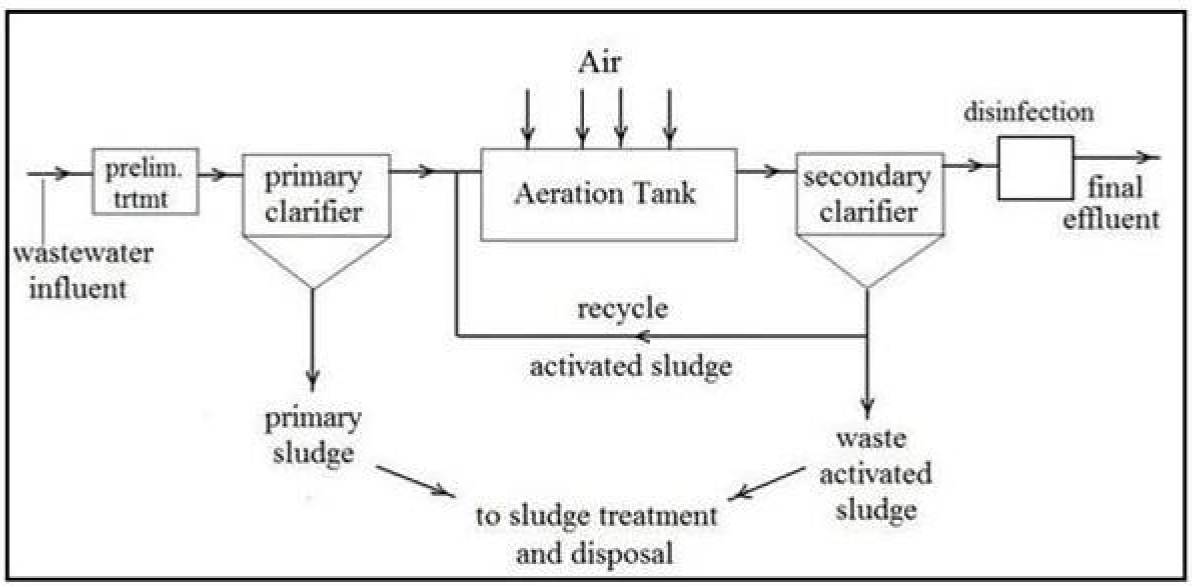


Figure 4-11 Flow diagram for a conventional AS plant giving high degree of treatment

From the primary sedimentation tank, the sewage flows to the aeration tank and is mixed with the activated sludge. The aeration tanks or aeration chambers are normally rectangular tanks 3 to

4.5m deep and about 4 to 6m wide. The length may range between 20 to 200m and the detention period between 4 to 8 hours for municipal sew ages. Air is continuously introduced into these tanks.

Methods of Aeration: There are two basic methods of introducing air into the aeration tanks,

(1) Diffused air aeration or Air diffusion
(2) Mechanical aeration

(1) Diffused Air Aeration

In the diffused air aeration method, compressed air under a pressure of 35 to 70kN/m^2 (0.35 to 0.7kg/cm^2) is introduced into the aeration chamber through diffusion plates or other devices are called diffusers. The main criteria for selection of a particular diffuser are that it should be capable of diffusing air in small bubbles, so as to provide the greatest possible efficiency of aeration. Porous plates and porous tubes made of quartz or crystalline alumina (Aluminum oxide) are generally used as diffusers. Plates are generally square in shape with dimensions of 30cm x 30cm and they are usually 25mm thick. These plates are fixed at the bottom of aeration tanks. Tube diffusers are generally 60cm long with internal diameter of 75mm and thickness of wall equal to 15mm. These tubes are suspended in the aeration tank, and can be taken out for cleaning, without emptying the tank. The effective areas for the above standard plate and tube diffusers work out to 780cm^2 and 1160cm^2, respectively.

Two types of aeration tanks are generally used. In one design, the tank is formed into a succession of ridges and furrows (Figure 4-12) and air is forced upward through diffuser plates placed at the bottom of the furrows. Such a tank is called the ridge and furrow type of aeration tank.

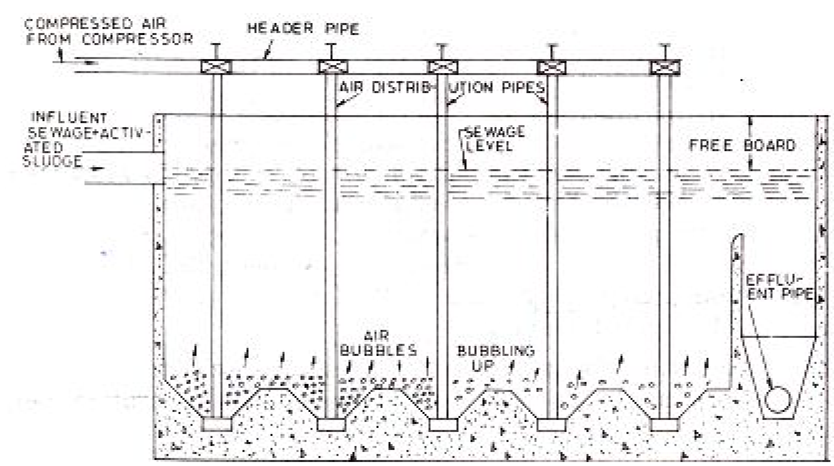


Figure 4-12 L-section of a ridge and furrow type of an aeration tank

Another popular design is the spiral flow type of aeration lank (Figure 4-13). In this tank, air is introduced near the side of the tank in such a way that spiral flow results in the tank; as shown.

The compressed air, in such a tank, can be supplied either through a plate diffuser or a tube diffuser, although tube diffusers are most widely used. This type of tank requires small quantity of compressed air at low pressure. Spiral motion set up by the compressed air, released through the tube diffusers, causes the required aeration.

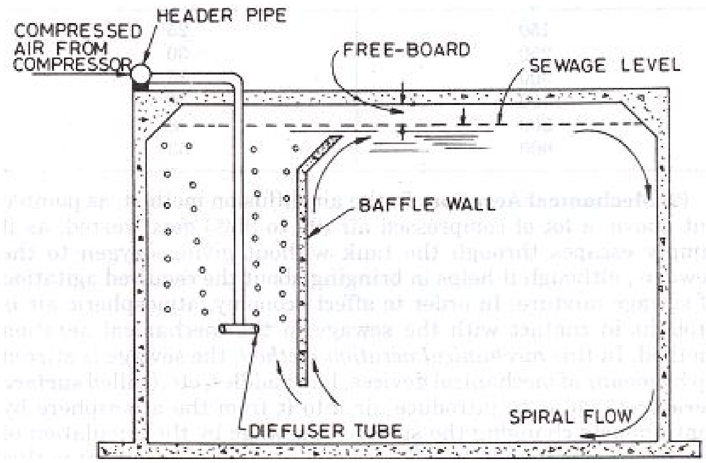


Figure 4-13 Cross-section of a spiral flow type of an aeration tank

Quantity of air required in diffused aerators:

In order to determine the capacity of the air compressor, it is necessary to determine the quantity of air that will be required. On an average, it may be assumed that about 4000 to 8000m^3 of free air will be required per million liters of sewage being treated. But this provision depends on the strength of sewage and various other factors. With respect to the BOD removal, the usual rate adopted is 100m^3/day of air per kg of BOD removed. Since only about 5% of oxygen in the air is actually involved in the biochemical action, the modern practice in design is to rationally calculate the oxygen requirement of the given wastewater and select aerator accordingly.

Volume of returned activated sludge:

The volume of returned activated sludge from secondary clarifier to the aeration tank mainly depends upon the extent of BOD desired to be removed. It is usually expressed as percentage of flow of sewage as $\frac{Q_R}{Q}$ where QR is the returned sludge rate in m^3/d and Q is the sewage inflow rate in m^3/d. Simple variation with the extent of desired BOD removal is shown in Table 4-3 below:

Table 4-3 Variation with the extent of BOD removal

Extent of BOD removal desired in ppm	Quantity of returned sludge as percentage of sewage flow
150	25
250	30
400	35

300	40
500	48
600	53

(2) Mechanical Aeration

The air-diffusion method, as pointed out above, a lot of compressed air (90 to 95%) gets wasted, as it simply escapes through the tank without giving oxygen to the sewage; although it helps in bringing about the required agitation of sewage mixture. In order to affect economy, atmospheric air is brought in contact with the sewage in the mechanical aeration method. In this mechanical aeration method, the sewage is stirred up by means of mechanical devices, like paddles, etc. (called surface aerators) ; so as to introduce air into it from the atmosphere by continuously changing the surface of sewage by the circulation of sewage from bottom to top. The only important requirement in this method is to have thorough agitation of sewage, so as to bring it in intimate contact with the atmosphere.

The aeration period depends on the mechanical process adopted for agitation. It generally varies between 6 to 8 hours. The quantity of the returned sludge, in mechanically aerated aeration tanks, is usually about 25 to 30 percent of the flow of sewage.

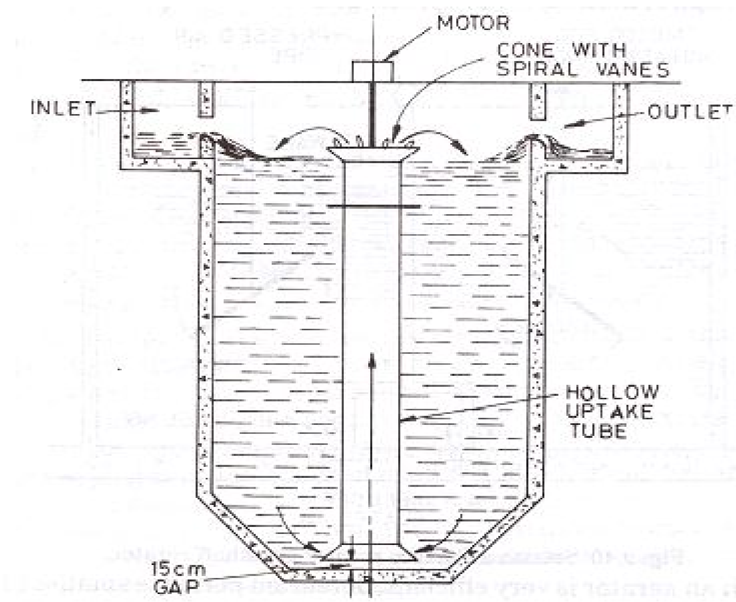


Figure 4-14 Section of a mechanical aerator

Sometimes a rectangular tank may also be used; but in that case, it must be divided suitably, into square units. At the centre of the tank, a hollow uptake tube is suspended from the top with a distance of about 15cm from the bottom of the tank. A steel cone with spiral vanes is provided at the top of the uptake tube and it is driven by a motor placed at the top of the tank. The cone is revolved at a high speed (60 rpm) which sucks the mixed liquor through the uptake tube by creating suction at the bottom, and sprays it at the surface towards the sides in the tank.

Numerous air bubbles are formed in this process, which bring about satisfactory aeration of sewage. Sewage also gets thoroughly mixed up with the activated sludge during its downward journey.

The mechanical aerators have gained considerable popularity in recent years. Their simplicity and ease of maintenance has resulted in their increasing popularity, especially for smaller plants. They are, however, sensitive even to slight variations in water levels.

3. Secondary Sedimentation Tank of an Activated Sludge Plant

From the aeration tank, the sewage flows to the final sedimentation tank. This tank will normally be of the general type (like a primary tank), with certain modifications. Since, there are no floating solids here; provisions for the removal of scum or floatage are not needed. The suspended particles in the aeration tank effluent are light in weight, and are thus markedly influenced by currents. Therefore, in these secondary settling tanks, a considerable length of overflow weir is desirable, to reduce the velocity of approach. Good design should provide a, weir overflow rate, not, exceeding 150m^3/day per lineal meter of weir. This value is based on average flow of sewage and not to the mixed liquor flow.

Solids loading are another important factor, which governs the design of secondary basin. This is because of the fact that in the secondary tank, hindered settling occurs and hence the settling velocity of discrete particles may not govern its design, as in the case of a primary sedimentation tank. The solids loading rate based on mixed liquor flow to the settling tanks, may be kept at about 100-150 kg/m^2 per day at average flow, and should not exceed 250 kg/m^2 per day at peak flows. Such rates ensure adequate sludge thickening and concentrated sludge returns.

The surface area for activated sludge settling tanks should be designed for both overflow rate and solids loading rate, and larger value adopted.

The detention period for such a sedimentation tank may be kept between $1\frac{1}{2} - 2$ hours, as the same is usually found to give optimum results.

The length to depth ratio of these tanks may be kept at about 5 for circular tanks, and 7 for rectangular ones. The depth may be kept in the range of 3.5 to 4.5m.

Since final settling is always required in an activated sludge plant, so as to provide the return activated sludge, duplicate secondary settling tanks are generally considered necessary.

Design considerations involved in an activated sludge plant

1. Aeration Tank Loadings

The important terms which define the loading rates of an activated sludge plant, include:

(i) Aeration Period (i.e. Hydraulic Retention Time - HRT)
(ii) BOD loading per unit volume of aeration tank (i.e. volumetric loading)
(iii) Food to Micro-organism Ratio (F/M Ratio)
(iv) Sludge age

(i) The Aeration Period or HRT

The aeration period (t) empirically decides the loading rate at which the sewage is applied to the aeration tank. For continuous flow aeration tank, this value is determined in the same manner as it is determined for an ordinary continuous sedimentation tank as:

$$\text{Detention period} = \frac{\text{Volume of the tank}}{\text{Rate of sewage flow in the tank}}$$

$$= \frac{\text{V in m}^3}{\text{Q in m}^3/\text{d}}$$

$$t = \frac{V}{Q} * 24 \text{ hour} \qquad 4.24$$

Where, t = aeration period in hours

V = Volume of aeration tank

Q = Quantity of wastewater flow into the aeration tank excluding the quantity of recycled sludge

(ii) Volumetric BOD Loading

Another empirical loading parameter is volumetric loading, which is defined as the BOD_5 1oad applied per unit volume of aeration tank. This loading is also called organic loading.

Volumetric BOD loading or Organic loading

$$= \frac{\text{Mass of BOD applied per day to the aeration tank through influent sewage in gm}}{\text{Volume of the aeration tank in m}^3}$$

$$= \frac{Q * Y_o \text{ (gm)}}{V \text{ (m}^3)} \qquad 4.25$$

Where, Q = Sewage flow into the aeration tank in m^3

Y_o = BOD_5 in mg/l (or gm/m^3) of the influent sewage

V = Aeration tank volume in m^3

This loading is quite similar to the BOD loading rate per cum of filter volume, as used and adopted in biological filtration.

(iii) Food (F) to Micro-organisms (M) Ratio

F/M ratio is an important rational organic loading rate adopted for an activated sludge process. It is a manner of expressing BOD loading with regard to the microbial mass in the system. The BOD load applied to the system in kg or gm is represented as food (F), and the total microbial suspended solid in the mixed liquor of the aeration tank is represented by M.

$$F/M \text{ ratio} = \frac{\text{Daily BOD load applied to the aerator system in gm}}{\text{Total microbial mass in the system in gm}}$$

If Y_o (mg/l) represents the 5 day BOD of the influent sewage flow of Q m^3/day, then eventually,
The BOD applied to the Aeration system = Y_o mg/l or gm/m^3
Therefore, BOD load applied to the aeration system

$$= F = Q * Y_o \text{ gm/day} \qquad \textbf{(i)}$$

The total microbial mass in the aeration system (M) is computed by multiplying the average concentration of solids in the mixed liquor of the aeration tank called Mixed Liquor Suspended Solids (MLSS) with the volume of the aeration tank (V).

$$M = MLSS * V$$
$$= X_t * V \qquad \textbf{(ii)}$$

Where, X_t is MLSS in mg/l
Dividing (i) by (ii), we get

$$\frac{F}{M} \text{ratio} = \frac{F}{M} = \frac{Q}{V} * \frac{Y_o}{X_t} \qquad 4.26$$

F/M ratio for an activated sludge plant is the main factor controlling BOD removal. The lower the F/M value the higher will be the BOD removal in the plant. The F/M ratio can be varied by varying the MLSS concentration in the aeration tank.

(iv) Sludge Age

The sludge age is an operation parameter related to the F/M ratio. It may be defined as the average time for which particles of suspended solids remain under aeration. It, thus, indicates the residence time of biological solids in the system. While aeration periods (i.e. liquid retention times) may be as short as 3 to 30h the residence time of biological solids in the system is much greater and is measured in days.

While sewage passes through the aeration tank only once and rather quickly, the resultant biological growths and the extracted waste organics (solids) are repeatedly recycled from the secondary clarifier back to the aeration tank, thereby increasing the retention time of solids. This time is called Solids Retention Time (SRT) or Mean Cell Residence Time (MCRT) or Sludge Age.

The most common method of expressing sludge age usually represented by θ_c in days, is to express it as the ratio of the mass of MLSS in the aeration tank relatively to the mass of suspended solids leaving the system per day.

Sludge age (θ_c)

$$= \frac{\text{Mass of suspended solids (MLSS) in the system (M)}}{\text{Mass of solids leaving the system per day}} \qquad 4.27$$

For a conventional activated sludge plant with the flow (Q), concentrations of solids (X_t), and BOD_5 (Y), as marked in Figure 4-15,

We can easily write:

(a) Mass of solids in the reactor

$$= M = V * (MLSS)$$

$$= V * X_t$$

Where, X_t is MLSS in the aeration tank (in mg/l).

(b)

(i) Mass of solids removed with the wasted sludge per day

$$= Q_w * X_R \qquad (i)$$

(ii) Mass of solids removed with the effluent per day

$$= (Q - Q_w) * X_E \qquad (ii)$$

Therefore, (b) Total solid removed from the system per day

= (i) + (ii)

$$= Q_w * X_R + (Q - Q_w) * X_E$$

Thus:

$$\text{Sludge age} = \theta_c = \frac{V * X_t}{Q_w * X_R + (Q - Q_w) * X_E} \qquad 4.28$$

Where, X_t = Concentration of solids in the influent of the aeration tank called the MLSS, mg/l

V = Volume of Aerator

Q_w = Volume of wasted sludge per day.

X_R = Concentration of solids in the returned sludge or in the wasted sludge in mg/l

Q = Sewage inflow per day

X_E = Concentration of solids in the effluent in mg/l

When the value of X_E (suspended solids concentration in the effluent of activated sludge plant) is very small, then the term $((Q - Q_w) * X_E$ in the above equation can be ignored, leading to:

$$\theta_c = \frac{V * X_t}{Q_w * X_R} \qquad 4.29$$

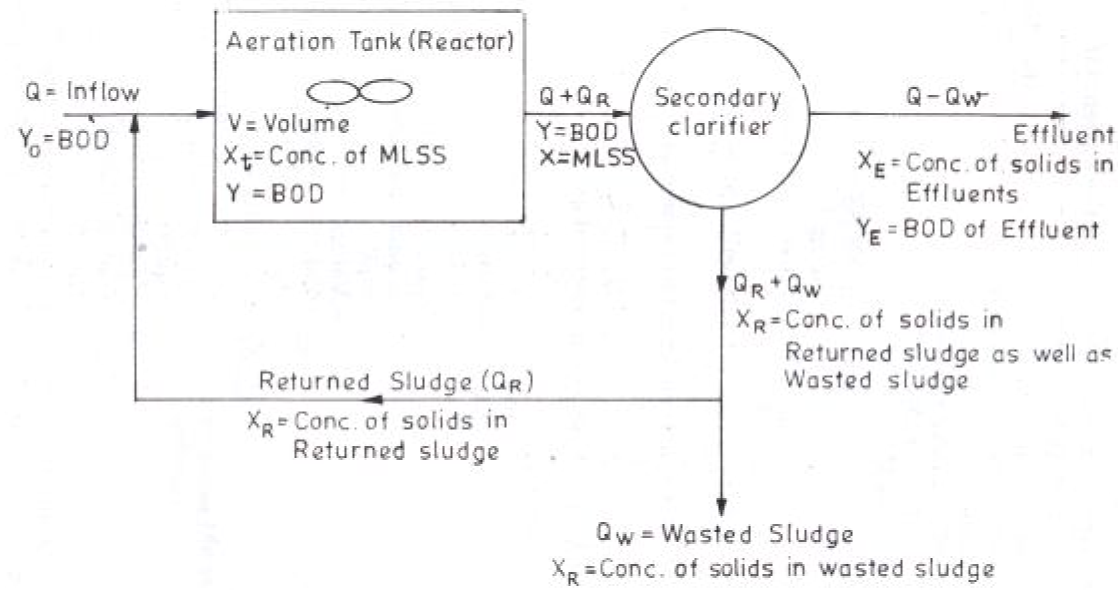


Figure 4-15 Flow chart of conventional activated sludge plant

In addition to using sludge retention time (θ_c) as a rational loading parameter, another rational loading parameter which has found wider acceptance is the specific substrate utilization rate (q) per day, and is defined as:

$$q = Q * \left(\frac{Y_o - Y_E}{V * X_t}\right) \qquad 4.30$$

Under steady state operation, the mass of wasted activated sludge is further given by

$$Q_w * X_R = y * Q(Y_o - Y_E) - K_e * X_t * V \qquad 4.31$$

Where, y = maximum yield coefficient

$$= \frac{\text{microbial mass synthesized}}{\text{mass of substrate utilized}}$$

K_e = Endogenous respiration rate constant (per day)

The values of y and K_e are found to be constant for municipal waste waters, their typical values being:

y = 1.0 with respect to TSS (i.e. MLSS)

= 0.6 with respect to VSS (i.e. MLVSS)

K_e = 0.06 (per day)

From the above equations, we can also work out as:

$$= \frac{1}{\theta_c} = yq - K_e \qquad 4.32$$

Since both y and K_e are constants for a given wastewater, it becomes necessary to define either θ_c or q.

The θ_c value adopted for the design controls the effluent quality and settleability and drainability of the biomass. Other parameters which are affected by the choice of θ_c values are oxygen requirement and quantity of waste activated sludge.

Example 4-3

An average operating data for conventional activated sludge treatment plant is as follows:
(1) Wastewater flow, Q = 35000m^3/d
(2) Volume of aeration tank, V = 10900m^3
(3) Influent BOD, Y_o = 250mg/l
(4) Effluent BOD, Y_E = 20 mg/l
(5) Mixed liquor suspended solids (MLSS), X_t = 2500mg/l
(6) Effluent suspended solids, X_E = 30mg/l
(7) Waste sludge suspended solids, X_R = 9700mg/l
(8) Quantity of waste sludge, Q_w = 220m^3/d

Based on the information above data, determine:
(a) Aeration period (hrs)
(b) Food to microorganism ratio (F/M) (kg BOD per day/kg MLSS)
(c) Percentage efficiency of BOD removal
(d) Sludge age (days)

Solution

(a) Aeration period (t) in hr is given by Eq. (9.41) as

$$t = \frac{V}{Q} * 24 = \frac{10{,}900}{35{,}000} * 24 = 7.47\ \text{h} \approx 7.5\ \text{h}$$

(b) F/M ratio

F = Mass of BOD removed

$$= Q * Y_o = 35000 * 250\ \text{gm/day} = \frac{35000 \times 250}{1000}\ \text{kg/day} = 8750\ \text{kg/day}$$

M = Mass of MLSS

$$= V * X_t = 10900\text{m}^3 * 2500\ \text{gm/m}^3 = \frac{10900 * 2500}{1000}\ \text{kg} = 27{,}250\text{kg}$$

$$\text{F/M ratio} = \frac{8750}{27{,}250} = 0.32\ \text{kg BOD per day/kg of MLSS}$$

(c) Percentage efficiency of BOD removal

$$= \frac{\text{Incoming BOD} - \text{Outgoing BOD}}{\text{Incoming BOD}} = \frac{250 - 20}{250} * 100\% = 92\%$$

(d) Sludge age in days (θ_c) is given by Eq. (9.48) as:

$$\theta_c = \frac{V * X_t}{Q_w * X_R + (Q - Q_w) * X_E}$$

$$= \frac{27250\text{ kg}}{\left(\frac{220\text{m}^3}{\text{d}} * \frac{9700\text{mg}}{\text{l}}\right) + \left(\frac{35000\text{m}^3}{\text{d}} - \frac{220\text{m}^3}{\text{d}}\right) * \frac{30\text{mg}}{\text{l}}}$$

$$= \mathbf{8.58\ days}$$

2. Sludge Volume Index (SVI)

The term sludge volume index or sludge index is used to indicate the physical state of the sludge produced in a biological aeration system. It represents the degree of concentration of the sludge in the system, and hence decides the rate of recycle of sludge (Q_R) required to maintain the desired MLSS and F/M ratio in the aeration tank to achieve the desired degree of purification.

SVI is defined as the volume of sludge occupied in ml by one gm of solids in the mixed liquor after settling for 30 minutes, and is determined experimentally.

The standard test which is performed in the laboratory to compute SVI of an aeration system involves collection of one liter sample of mixed liquor from the aeration tank from near its discharge end in a graduated cylinder. This 1 liter sample of mixed liquor is allowed to settle for 30 minutes and the settled sludge volume (V_{ob}) in ml is recorded as to represent sludge volume. This volume V_{ob} in ml per liter of mixed liquor will represent the quantity of sludge in the liquor in ml/l.

The above sample of mixed liquor after remixing the settled solids is further tested in the laboratory for MLSS by the standard procedure adopted for measuring the suspended solids in sewage. Let this concentration of suspended solids in the mixed liquor in mg/l be X_{ob}. Then SVI is given by the equation

$$\text{SVI} = \frac{\text{Vob (ml/l)}}{\text{Xob (mg/l)}} = \frac{V_{ob}}{X_{ob}}\text{ml/mg} \qquad 4.33$$

$$\text{SVI} = \frac{V_{ob}}{X_{ob}} * 1000\text{ml/g}$$

The usual adopted range of SVI is between 50 - 150 ml/gm and such a value indicates good settling sludge.

Note: When the given SVI value in ml/gm is divided by $10^3 * 10^3$ (i.e. 10^6), we will get SVI value in l/mg. SVI value in l/mg will therefore be $\frac{SVI}{10^6}$. SVI value in mg/l will, thus, be given by $\frac{10^6}{SVI}$.

Sludge Recycle and Rate of Return Sludge

The MLSS concentration in the aeration tank is controlled by the sludge recirculation rate and the sludge settleability and thickening in the secondary sedimentation tank. The relationship between sludge recirculation ratio $\frac{Q_R}{Q}$ with X_t (MLSS in tank) and X_R (MLSS in returned or wasted sludge) is given as:

$$\frac{Q_R}{Q} = \frac{X_t}{X_R - X_t} \qquad 4.34$$

Where, Q_R = Sludge recirculation rate in m^3/d

X_t = MLSS in the aeration tank in mg/l

X_R = MLSS in the returned or wasted sludge in mg/l

The settleability of sludge is determined by sludge volume index (SVI), which is determined in the laboratory.

If it is assumed that the sedimentation of suspended solids in the laboratory is similar to that in the sedimentation tank, then:

$$X_R = \frac{10^6}{SVI} \qquad 4.35$$

where, SVI value in mg/l

Then,

$$\frac{Q_R}{Q} = \frac{X_t}{\frac{10^6}{SVI} - X_t} \qquad 4.36$$

Values of return sludge ratios adopted in different types of activated sludge systems are shown in Table 4-4. Its value for conventional sludge plant varies between 0.25 and 0.50.

The return sludge has always to be pumped and the pump capacity should be designed for a minimum return sludge ratio of 0.50 to 0.75 for large plants and 1.0 to 1.5 for smaller plants irrespective of the theoretical requirement. The required capacity should be provided in multiple units to permit variation of return sludge ratio as found necessary during the operation of the plant.

Wasting of Excess Sludge (Q_w)

We know that the sludge generated in the aeration tank has to be partly discharged and wasted out of the plant to maintain a steady level of MLSS in the system. The excess sludge quantity will increase with the increasing F/M ratio and decrease with temperature. In the case of domestic sewage, Q_w will be about 0.50 - 0.75kg per kg BOD removed for the conventional sludge plants (having F/M ratio varying between 0.4 and 0.3).

Excess sludge may be wasted either from the sludge return line or directly from the aeration tank as mixed liquor. The latter procedure is usually preferred since the concentration of suspended solids will then be fairly steady in the waste discharge making the control easy.
In conventional plants, the wasted sludge is taken directly to a sludge thickner and digester or to the primary settling tank for its disposal along with the primary sludge. In extended aeration plants, however, the excess sludge is directly taken to the sludge drying beds.

Modifications of the Basic Activated Sludge Process

In the basic activated sludge process also called conventional aeration process, the re-circulated activated sludge is added to the inlet end of the aeration tank as a single dose. The regime flow employed in the aeration tank is plug flow and not mixed flow. Plug flow implies that the sewage moves down progressively along the aeration tank essentially unmixed with the rest of the tank contents. The other type of flow regime called complete mixed flow involves the rapid dispersal of the incoming sewage throughout the tank and is adopted in the extended aeration process.
In a conventional aeration tank (of plug flow type), the F/M ratio and the oxygen demand will be the highest at the inlet end, and will then progressively decrease. In the complete mix system on the other hand, the F/M ratio and oxygen demand will be uniform throughout the tank.
The plug flow regime is achieved in such an activated process by employing a long and narrow configuration of the aeration tank with length equal to 5 to 50 times the width. The sewage and the mixed liquor are let in at the head of the tank and withdrawn at its end. Because of the plug flow regime, the oxygen demand at the head of the aeration tank is high and then tapers down. However, air is supplied in the process at a uniform rate along the length of the tank. This leads to either oxygen deficiency in the initial zone or wasteful application of air in the subsequent reaches.
The conventional system is always preceded by primary settling. The plant itself consists of an aeration tank, a secondary settling tank, a sludge return line and an excess sludge waste line leading to digester. The BOD removal in this process is 85 - 92%.
The main limitations of the conventional system are that:

(i) the aeration tank volume requirement is high;
(ii) there is a lack of operational stability at times of excessive variation in the rate of inflow or its BOD strength

In order to overcome such difficulties posed by a conventional system plant, and to meet specific treatment objectives, several modifications of the conventional system have been suggested by modifying the process variables.

The important modified processes are:

- (i) Tapered aeration process;
- (ii) Step aeration process;
- (iii) Contact stabilization process;
- (iv) Complete mix process;
- (v) Modified aeration process;
- (vi) Extended aeration process

In spite of its various limitations, the conventional system for historical reasons, is the most widely used type of the activated sludge process. Plants up to 300MLD capacity have been built in diameter. In addition to conventional activated sludge plants, the complete mixed plants and the extended aeration plants have also been found a wider acceptance in modern days, particularly for obtaining high BOD removals in smaller capacity plants.

i. Tapered Aeration Process

This process involves a very little modification of the conventional process, and ensures higher air supply at the inlet and in the initial length of the tank, as compared to the downstream length. The process is surely based on the fact that as the mixed liquor progresses through the aeration tank, its air requirement goes on reducing. Therefore, in a tapered aeration plant, compressed air is supplied at higher rates near the inlet end of the tank, and is gradually decreased as sewage moves towards the outlet end of the tank. Such a process therefore helps us in ensuring optimal application of air in the aeration tank.

Ordinarily, 45% of air is supplied to the first one-third length of the tank, 30% to the second one-third length of the tank, and the rest 25% to the remaining one-third length of the tank.

Number of diffuser plates is thus varied accordingly. Such a modification to the conventional activated plants using diffused air aeration, has now-a-days become a common feature, and is invariably adopted in all modern designs. The loading parameters of such a plant do not materially differ from a conventional one, and are given in Table 4-4.

ii. Step Aeration Process

In the step aeration process, the sewage is introduced along the length of the aeration tank in several steps, while the return sludge is introduced at the head, as shown in Figure 4-16.

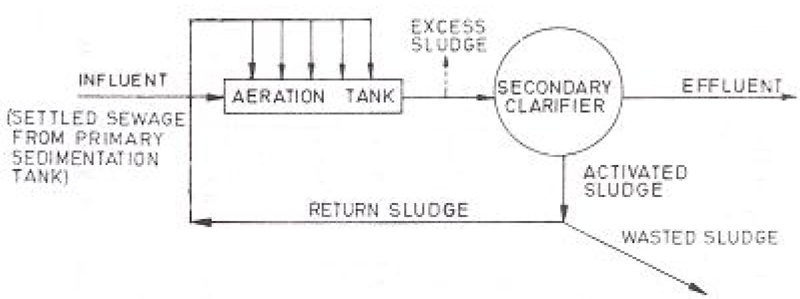


Figure 4-16 Flow chart of step-aeration process

Such an arrangement results in a uniform air requirement along the entire length of the tank, and hence the uniform air supply of the conventional plants, can be efficiently used. The process enables an appreciable reduction in the aeration tank volume, without lowering the BOD removal efficiency. Step aeration method has considerable capacity to absorb shock organic loadings. The method has found application for larger plants of capacities up to about 1000MLD.
The loading parameters of such a plant are given Table 4-4.

iii. Contact Stabilization Process or Biosorption Process

In this process, the sewage and recycled or returned sludge are mixed and aerated for a comparatively shorter period of 0.5 to 1.5 hour in a special mixing tank, called contact tank. This mixing will allow the suspended and dissolved organic matter to be sorbed to the activated sludge floc. The sorbed organics and flocs are removed in the secondary settling tank, where the effluent from the contact tank enters. These settled sorbed organics and flocs are then transferred to a sludge aeration tank (called stabilization tank or **aerodigester**) where the organics are stabilized over a period of about 3 to 6 hours before it is fed back into the contact aeration tank. The stabilized sludge is then mixed with the influent wastewater again, and the process is repeated. The flow diagram for this process is shown in Figure 4-17, and the loading parameters are given in Table 4-4.

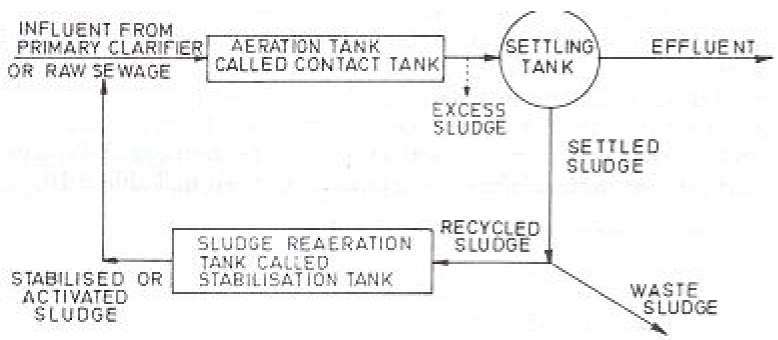


Figure 4-17 Contact-stabilization process

The contact stabilization process is quite effective in the removal of colloidal and suspended organic matter, but it is not very effective in removing soluble organics. The method is well suited for the treatment of fresh domestic sewage, containing only a low percentage of soluble BOD.
Compared to the conventional system, the contact stabilization process has greater capacity to handle shock organic loadings, because of the biological buffering capacity of the sludge reaeration tank. The process also presents greater resistance to toxic substances in the sewage as the biological mass is exposed to the main stream of sewage containing the toxic constituents only for a short time.

The air requirements of the process are the same as for the conventional system, the air supply being divided equally between the contact aeration tank and the sludge re-aeration tank. However, the total aeration tank volume required for both the aeration tanks is only about half of the volume required in the single conventional sludge aeration tank. The process therefore presents an effective method of up rating the existing conventional plants where sewage characteristics are satisfactory. Moreover, the total aeration time is considerably reduced and the plant capacity is thereby increased.

The process has found application in medium sized plants with capacities up to 40 MLD.

iv. Complete Mix Process

Complete mix activated sludge plants were developed particularly for smaller cities, where the hourly variations in sewage were quite high, and as such conventional plants were experiencing serious problems of biological instability.

In such a plant, the plug flow regime of a conventional plant is replaced by a completely mixed flow regime. Such a flow regime can be achieved by thorough mixing of sewage and return sludge. Sewage and return sludge are therefore distributed uniformly along one side of the aeration tank, and the aerated sewage is withdrawn uniformly along the opposite side. Mechanical aerators are installed in the centre of a circular or square aeration tank, which may be used in such a plant. Such mechanical aerators must have adequate mixing capacity to ensure thorough mixing of sewage and return sludge. The flow chart for such a plant is shown in Figure 4-18 and the loading parameters of such a plant are given in Table 4-4.

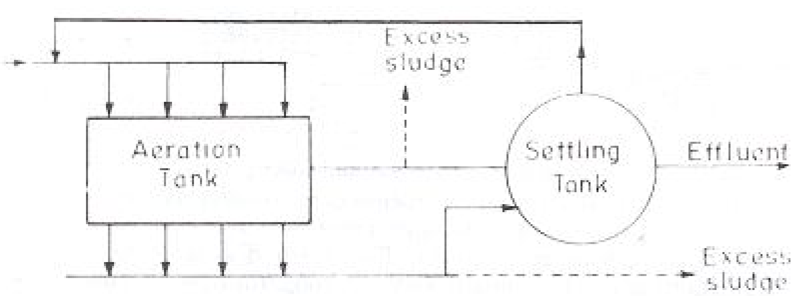


Figure 4-18 Flow chart of a complete mix plant

The complete mix plant possesses capacity to hold much higher MLSS concentration level in the aeration tank; say 3000 to 4000mg/l as against 1500 to 3000 mg/l of a conventional plant. This helps in adopting smaller volume for the aeration tank. The plant has an increased operational stability at shock organic loadings, and also increased capacity to treat toxic biodegradable wastewaters like phenols. Such a plant is less liable to upsets by slugs of flows of toxic wastewaters.

Such plants have been widely used for smaller plant capacities of less than 25 MLD, particularly for the towns where municipal and industrial wastewaters flow together.

v. Modified Aeration Process

When an intermediate quality of effluent containing higher BOD is permissible, such as at a place where effluent is to be used for farming, this modified aeration plant may be adopted, as it leads to substantial savings in construction and aeration costs.

Such a process does not need any primary sedimentation tank, as is invariably required in a conventional plant. The process ensures short aeration period, high volumetric loading, high F/M ratio, low percentage of sludge return, low concentration of MLSS, as reflected in Table 4-4. The BOD removal is also low say only 60 - 75% or so. The process has been employed mainly in large plants with capacities above 200 MLD.

vi. Extended Aeration Process

The flow scheme of an extended aeration process and its mixing regime are similar to that of the complete mix process. Primary sedimentation is frequently avoided in this process, but grit chamber or comminutor is often provided for screenings.

As its name suggests, the aeration period is quite large and extends to about 12 - 24 hours, as compared to 4 to 6 hours in a conventional plant. The loading parameters for such a process are already given in Table 4-4. The process permits low organic loading, high MLSS concentration, and low F/M ratio. The BOD removal efficiency is also quite high, to say about 95 - 98% as compared to 85 - 92% of a conventional plant.

The air or oxygen requirement is of course quite high, which increases the running cost of the plant considerably. The plant, however, offers another advantage as no separate sludge digester is required here, because the solids undergo considerable endogenous respiration and get well stabilized over the long detention periods adopted in the aeration tank. The sludge produced is, thus, capable to be directly taken to the sludge drying beds. Also, the excess sludge production is minimum. The operation is also simpler due to the elimination of primary settling and separate sludge digestion. Such a process is quite suitable for small communities having sewage flows of less than 4 MLD or so.

Table 4-4 Characteristics and design parameters of different activated sludge systems

Process type	Flow regime	MLSS mg/l	$\frac{MLVSS}{MLSS}$	$\frac{F}{M}$	HRT hrs	Volumetric Loading kg BOD_5 per m^3	SRT (days) θ_c	$\frac{Q_R}{Q}$ Return Sludge ratio	BOD removal percent	kg O_2 reqd. per kg BOD_5 removed	Air requirement in m^3 per kg of BOD_5 removed
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Conventional	Plug	1500 to 3000	0.8	0.4 to 0.3	4 to 6	0.3 to 0.7	5 to8	0.25 to 0.5	85 to 92	0.8 to 1.0	40 to 100
Taperd aeration	Plug	1500 to 3000	0.8	0.4 to 0.3	4 to 6	0.3 to 0.8	5 to 8	0.25 to 0.5	85 to 92	0.7 to 1.0	50 to 75
Step aeration	Plug	2000 to 3000	0.8	0.4 to 0.3	3 to 6	0.7 to 1.0	5 to 8	0.25 to 0.75	85 to 92	0.7 to 1.0	50 to 75^{+}
Contact stabilization	Plug	1000 to 3000* 3000 to 6000**	0.8	0.5 to 0.3	0.5 to 1.5* 3 to 6**	1.0 to 1.2	5 to 8	0.25 to 1.0	85 to 92	0.7 to 1.0	50 to 75
Complete mix	Complete mix	3000 to 4000	0.8	0.5 to 0.3	4 to 5	0.8 to 2.0	5 to 8	0.25 to 0.8	85 to 92	0.8 to 1.0	50 to 75
Modified aeration	Plug	300 to 800	0.8	3.0 to 1.5	1.5 to 3	1.2 to 2.4	0.2 to 0.5	0.05 to 0.15	60 to 75	0.4 to 0.6	25 to 50
Extended aeration	Complete mix	3000 to 5000	0.5 to 0.6	0.18 to 0.1	12 to 24	0.2 to 0.4	10 to 25	0.5 to 1.0	95 to 98	1.0 to 1.2	100 to 135

* in contact aeration

** in sludge aeration tank

+ divided equally between contact aeration tank and sludge re-aeration tank

Size and Volume of the Aeration Tank

Equations (4.28) and (4.31) can also be combined to yield:

$$V * X_t = \frac{y * Q(Y_o - Y_E)\theta_c}{1 + K_e} \qquad 4.37$$

This equation can be used to calculate the volume of the aeration tank (V) for an assumed value of X_t (MLSS concentration in aeration tank) and a selected value of θ_c. Alternatively, the tank volume can be determined from Eq(4.27) for an assumed value of F/M ratio and tank MLSS (X_t).

It can be seen that economy in reactor volume can be achieved by assuming a higher value of X_t. However, it is seldom taken to be more than 5000g/m^3 (i.e. mg/L). A common range is between 1000 to 4000 g/m^3 (see Table 4-4). Considerations which govern the upper limit are:

(i) initial and running cost of sludge recirculation system to maintain a high value of MLSS;

(ii) limitations of oxygen transfer equipment to supply oxygen at required rate in a small reactor volume;

(iii) increased solids loading on secondary clarifier which may necessitate a larger surface area to meet limiting solid flux;

(iv) design criteria for the tank and minimum HRT (t) for the aeration tank for stable operation under hydraulic surges

Except in the case of extended aeration plants and completely mixed plants, the aeration tanks are designed as long narrow channels. This configuration is achieved by the provision of round the-end baffles in small plants when only one or two tank units are proposed; and by constructing long and narrow rectangular tanks with common intermediate walls in large plants when several units are proposed.

In extended aeration plants (other than oxidation ditches) and in complete mix plants, the tank shape may, however, be kept circular or square when the tank capacity is small, and rectangular with several side inlets and equal number of side outlets when the plant capacity is large.

The width and depth of aeration channel for conventional plants depend upon the type of aeration equipment used. The depth controls the aeration efficiency and usually ranges from 3 to 4.5m. The higher value of depth of 4.5m is found to be more economical for plants of more than 50MLD capacity. Beyond 70 MLD, duplicate units are preferred. The width controls the mixing and is usually kept between 5 to 10 m. Width-depth ratio should be adjusted between 1.2 and 2.2. The length should not be less than 30m and not ordinarily longer than 100m in a single section length before doubling back. The horizontal velocity should be around 1.5m/min. Excessive width may lead to settlement of solids in the tank. Triangular baffles and fillets are used to eliminate dead spots and induce spiral flow in the tanks. The free-board in the tank is generally kept between 0.3 - 0.5m.

While designing the aeration tanks, due consideration should also be given to the need of emptying them for maintenance and repair of aeration equipment. Intermediate walls should be designed for empty conditions on either side. The method of dewatering should be considered in the design and provided for during construction.

The inlet and outlet channels of the aeration tank should be designed for empty conditions on either side. The method of dewatering should be considered in the design and provided for during construction. The inlet and outlet channels should be designed to maintain a minimum velocity of 0.2m/s to avoid deposition of solids. The channels or conduits and their appurtenances should be sized to carry the maximum hydraulic load to the remaining aeration tank units when anyone unit is out of operation.

The inlet should provide for free fall into aeration tank when more than one tank unit or more than one inlet is proposed. The free fall will enable positive control of the flows through the different inlets. Outlets usually consist of free fall weirs. The weir length should be sufficient to maintain a reasonably constant water level in the tank. When multiple inlets or multiple tanks are involved and the inlets should be provided with valves, gates or stop planks to enable regulation of flow through each inlet.

Oxygen Requirement of the Aeration Tanks

Oxygen is required in the activated sludge process in the aeration tank for oxidation of part of the influent organic matter, and also for endogenous respiration of the micro-organisms in the system.

The total oxygen requirement may be computed by using the equation

$$O_2 = \left(\frac{Q(Y_o - Y_E)}{f} - 1.42Q_w * X_R\right) \text{gm/day} \qquad 4.38$$

Where,

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

1.42 = oxygen demand of biomass in gm/gm

The above formula represents the oxygen demand for carbonaceous BOD removal and does not account for nitrification. The extra requirement of oxygen for nitrification is theoretically found to be 4.56kg O_2/kg NH_3 - N oxidized to NO_3 - N.

The total oxygen requirement per kg BODs removed for different activated sludge processes are given in Table 4-4 in col. (11). The amount of oxygen required for a particular process will increase with the range shown in the table, as the F/M value decreases.

The aeration facilities of the activated sludge plant shall be designed to provide the calculated oxygen demand of the waste water against a specific level of DO in the wastewater. The aeration devices, besides supplying the required oxygen demand, shall also provide adequate mixing or agitation, so that the entire MLSS present in the aeration tank will become available for the biological activity. The recommended DO concentration in the aeration tank is in the range of 0.1 to 1.0mg/L for conventional activated plants; and is in the range of 1 - 2mg/L for extended aeration type of activated plants; and shall be above 2mg/L when nitrification is required in the activated sludge plant.

Aerators are rated on the basis of the amount of oxygen (kg) that they can transfer to the tap water under standard conditions at 20°c, 760mm Hg barometric pressure and zero DO per unit of energy consumed.

The oxygen transfer capacity (N) under field conditions can be calculated from the standard oxygen transfer capacity) (Ns) by the formula:

$$N = \frac{N_s * (D_s - D_L)(1.024)^{(T-20°c)} * \alpha}{9.17} \qquad 4.39$$

Where, N = Oxygen transferred under field conditions in kg O_2/kWh

N_s = Oxygen transfer capacity under standard conditions in kg O_2/kWh

Ds = Dissolved oxygen-saturation value for sewage at operating temperature

D_L = Operation DO level in aeration tank, usually 1 to 2 mg/L

T = Temperature in °c.

α = Correction factor for oxygen transfer for sewage, usually 0.8 to 0.85

Oxygen may be supplied either by surface aerators or by diffused air aeration systems employing fine or coarse diffusers. In diameter, surface aerators are preferred because of easier maintenance. The oxygen transfer capacities of surface aerators, and fine and coarse diffused air systems, under standard conditions, lie between 1.2 - 2.4, 1.2 - 2 and 0.6 - 1.2kg O_2/kWh respectively.

Example 4-4

Design a conventional activated sludge plant to treat domestic sewage with diffused air aeration system, given the following data:

Population = 35000

Average sewage flow = 180 l/c/d

BOD of sewage = 220mg/l

BOD removed in primary treatment = 30%

Overall BOD reduction desired = 85%

Solution

Daily sewage flow

$$= Q = 180 * 35000 \text{ l/day} = 6300\text{m}^3\text{/day}$$

BOD of sewage coming to aeration

$$= Y_o = 70\% * 220\text{mg/l} = 154\text{mg/l}$$

BOD left in effluent

$$= Y_E = 15\% * 220\text{mg/l} = 33\text{mg/l}$$

BOD removed in activated plant

$$= 154 - 33 = 121\text{mg/l}$$

Efficiency required in Activated plant

$$= \frac{121}{154} = 0.79$$

From Table 4-4, for efficiency of 85 - 92%, we use F/M ratio as 0.4 to 0.3 and MLSS between 1500 and 3000. Since efficiency required is on lower side, we can use moderate figures for F/M ratio and MLSS.

So let us adopt F/M = 0.35

Similarly adopt MLSS (X_t) = 2000 mg/l

Using

$$\frac{F}{M} = \frac{Q * Y_o}{V * X_t}$$

$$35 = \frac{6300 * 154}{V * 2000}$$

V = volume of aeration tank

$$= \frac{6300 * 154}{2000 * 0.35} = 1386\text{m}^3$$

(i) Check for Aeration period or HRT (t)

Using

$$t = \frac{V}{Q} * 24\ h = \frac{1386}{6300} * 24\ h$$

$= 5.28\ h$ (within the limits of 4 to 6 h) ... ... ok

(ii) Check for SRT (θ_c)

From equation

$$V * X_t = \frac{y * Q(Y_o - Y_E)\theta_c}{1 + K_e}$$

Where, V = 1386m^3

X_t = 2000mg/l

y = yield coefficient = 1.0 with respect to MLSS

Q = 6300m^3/d

K_e = Endogenous respiration constant = 0.06d^{-1}

Y_o = BOD of influent in aeration tank = 154mg/l

Y_E = BOD of effluent = 33mg/l

Substituting the values, we get

$$1386 * 2000 = \frac{0.5 * 6300(154 - 33) * \theta_c}{1 + 0.06 * \theta_c}$$

$$1 + 0.06 * \theta_c = 0.275\theta_c$$

$$\theta_c = \frac{1}{0.215} = 4.65 \text{ days} \approx 5 \text{ days} \ldots \text{ ok! as it lies between 5 and 8 days}$$

(iii) Check for volumetric loading

Using equation

Volumetric loading = $\frac{Q * Y_o}{V}$ gm of BOD/m^3 of tank volume

$$= \frac{630 * 154}{1386} gm/m^3 = 700\ gm/m^3 = 0.7\ kg/m^3$$

(Within the permissible range of 0.3 - 0.7kg/m^3) …ok!

(iv) Check for Return sludge ratio (for SVI ranging between 50 - 150ml/gm

Let us take 100ml/gm.

Using equation

$$\frac{Q}{Q_R} = \frac{X_t \text{ (i.e. MLSS)}}{\frac{10^6}{SVI} - X_t}$$

Where, SVI = 100ml/gm

X_t = 2000mg/l

$$\frac{Q}{Q_R} = \frac{2000}{\frac{10^6}{100} - 2000} = 0.25$$

(i.e. within the prescribed range of 25 to 50%) …ok!

We will, for conservative purposes, however provide 33% return sludge, giving SVI = 125, ok. The sludge pumps for bringing recirculated sludge from the secondary sedimentation tank will thus have a capacity = 33% * Q = 33% * 6300m^3/d = 2100m^3/d.

Tank Dimensions

Adopt aeration tank of depth 3m and width 4.5m. The total length of the aeration channel required

$$= \frac{\text{Total volume required}}{B * D} = \frac{1386}{4.5 * 3} = 102.7\text{m, take } 105\text{m}$$

Provide a continuous channel, with 3 aeration chambers, each of 35m length. Total width of the unit, including 2 baffles each of 0.25m thickness = 3 * 4.5m + 2 * 0.25 = 14m. Total depth provided including free-board of 0.6m will be 3 + 0.6 = 3.6m.

Overall dimensions of the Aeration tank will be 35m * 14m * 3.6m.

Rate of Air Supply Required

Assuming the air requirement of the aeration tank to be 100m^3 of air per kg of BOD removed,

Air required i.e. blower capacity

$$= 100 * \frac{121 * 6300}{1000} = 53\text{m}^3/\text{min}$$

Let standard diffuser plates of 0.3m * 0.3m * 25mm size, releasing 1.2m^3 of air/min/m^2 with 0.3mm pores may be used. Then, the total No. of plates required

$$= \frac{53}{1.2 * 0.3 * 0.3} = 491; \text{ take } 500$$

Design of Secondary sedimentation Tank

Adopting a surface loading rate of 20m^3/day/m^2 at average flow of 6300m^3/day,

(i) Surface area required

$$= \frac{6300}{20} = 315\text{m}^2$$

Adopting a solids loading of 125kg/day/m^2 for MLSS of 2000mg/l,

(ii) the surface area required

$$= \frac{6300 * 2000}{1000} * \frac{1}{125} = 100.8\text{m}^2$$

The higher surface area of 315m^2 is adopted.

Adopting a circular tank,

$$\text{diameter of tank} = \sqrt{\frac{315 * 4}{\pi}} = 20\text{m}$$

Weir loading for a circular weir placed along the periphery of the tank having length 20 1t will be:

$$= \frac{6300}{20\pi}\text{m}^3/\text{day}/\text{m} = 100.3 < 150;\ \text{ok!}$$

Note: lf weir loading exceeds the permissible value; we may provide a trough instead of a single weir at the periphery.

Hence, provide 20 m diameter secondary settling tank.

Design of Sludge Drying Beds

In order to design sludge drying beds, the quantity of excess wasted sludge will be calculated by:

$$\theta_c = \frac{V * X_t}{Q_w * X_R}$$

$$4.65\ \text{d} = \frac{6300\text{m}^3/\text{d} * 2000\text{gm}/\text{m}^3}{Q_w * X_R}$$

$$Q_w * X_R = \frac{6300\text{m}^3/\text{d} * 2000\text{gm}/\text{m}^3}{4.65\ \text{d}} = 2800\text{kg}/\text{d}$$

For 10 kg/m^3 SS concentrating in secondary sludge, excess secondary sludge volume

$$= \frac{2800\text{kg}/\text{d}}{10\text{kg}/\text{m}^3} = 280\text{m}^3/\text{d}$$

Note: This secondary sludge volume of 280m^3/d shall be taken to sludge drying beds, along with the primary sludge. The volume of primary sludge can be calculated if the concentration of suspended solids in sewage is known along with knowing the degree of removal of suspended solids in primary settling. Since SS of sewage is not given in this question, the quantity of primary sludge cannot be worked out; and hence the design of sludge drying beds cannot to be done with the given data.

4.4.3 Rotating Biological Contractors

The Rotating Biological Contractor's process of secondary wastewater treatment has been recently developed and does not fit precisely in to either the trickling filter or the activated sludge categories, but does employ principle common to both of them.

A rotating biological contractor (CRBC) is a cylindrical media made of closely mounted thin flat circular plastic sheets or discs of 3 to 3.5m in diameter, 10mm thick, and placed at 30 to 40mm spacing mounted on a common shaft. Thinner materials can be used by sandwitching a corrugated sheet between two flat discs and welding them together as a unit, as shown in Figure 4-19:

Figure 4-19: Rotating Biological Contractors placed in series

The RBC's are usually made in up to 8m length, and may be placed in series or parallel in a specially constructed tank(s), through which the wastewater is allowed to pass. The RBC's are kept immersed in wastewater by about 40% of their diameter. The RBC's are rotated around their central horizontal shaft, at a speed of 1 - 2rpm by means of power supplied to the shaft. Approximately 95% of the surface area is thus alternately immersed in the wastewater and then exposed to the atmosphere above the liquid.

When the process is operated, the microorganisms of the wastewater begin to adhere to the rotating surfaces and grow there, until the entire surface area of the discs gets covered with 1 to 3mm layer of biological slime. As the discs rotate, they carry a film of wastewater into the air, where it trickles down the surface of the discs, absorbing oxygen. As the discs complete their rotation, this film mixes with the wastewater in the tank, adding to the oxygen of the tank and mixing the treated and partially treated wastewater. As the attached microorganisms pass through the tank, they absorb other organics for breakdown. The excess growth of microorganisms is sheared from the discs, as they move through the wastewater tank. The dislodged organisms are kept in suspension by the moving discs. This suspended growth finally moves down with the sewage flowing through the tank to a downstream settling tank for removal. The effluent obtained is of equal or even better quality than what is obtained from other secondary treatments. The quality of the effluent can further be improved by placing several contractors in series along the tank. The method can thus provide a high degree of treatment, including biological conversion of ammonia to nitrates.

As is evident, a given set of discs (i.e. an RBC) serves the following purposes:

(i) They provide media for buildup of attached microbial growth.

(ii) They bring the growth of microbes in contact with the wastewater.

(iii) They aerate the wastewater and the suspended microbial growth in the wastewater tank.

In this process, the attached growths are similar in concept to a trickling filter, except that here the microorganisms are passed through the wastewater, rather than the wastewater passing over the microbes, as happens in a trickling filter. This method realizes some of the advantages of both the trickling filter and the activated sludge process.

The sludge produced in the process contains about 95 - 98% moisture and may amount to about 0.4kg per kg of BOD_5 applied. The theoretical model of the process is similar to that for trickling filter, but actual design is still empirical and based on the data from the successful working plants and as developed by the process manufacturers.

The hydraulic loading rates may vary between 0.04 - 0.06m/day, and organic loading rates between 0.05 - 0.06kg BOD/m^2 per day, based upon the disc surface area. Sloughing of biological solids is more or less continuous and the effluent contains a relatively constant concentration. The solids settle well and clarifier surface overflow rates of about 33m^3/m^2 per day are reported to be satisfactory.

Kinetics in RBC

Kinetic equation of RBC is based on substrate removal:

$(\frac{1}{X}\frac{dS}{dt})$ = specific rate of utilization

$(\frac{dS}{dt})$ = rate of substrate utilization

k = rate constant

S = substrate concentration

Q = flow rate

S_o = influent substrate concentration

S_e = effluent substrate concentration

X = cell mass

A = disc area

$$-\frac{1}{X}\frac{dS}{dt} = kS \quad 4.40$$

$$\frac{dS}{dt} = Q(S_e - S_o)$$

$$-\frac{1}{X}Q(S_e - S_o) = kS_e$$

$$X \propto A$$

$$\frac{Q}{A}(S_o - S_e) = kS_e \quad 4.41$$

The term $\frac{Q}{A}(S_o - S_e)$ is equal to the rate of reaction, r.

Thus,

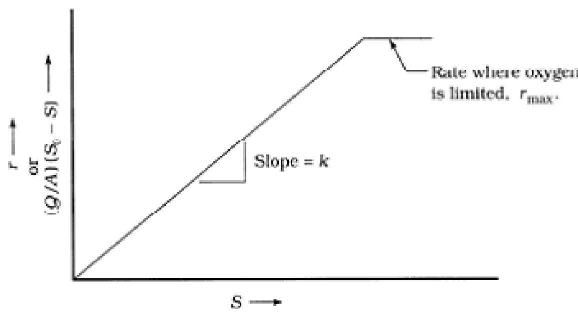
$$\frac{Q}{A}(S_o - S_e) = r = kS \quad 4.42$$

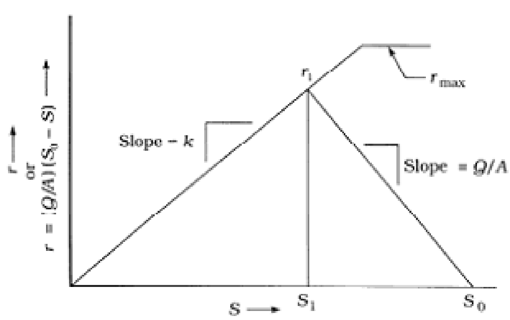
This equation is in the form of (y = mx), so it can be graphically presented as shown in the figure below. Rearranging above equation:

$$\frac{Q}{A} = \frac{r}{S_o - S_1} = \text{slope} \quad 4.43$$

For a series of contactors (n contactors)

$$\frac{Q}{A} = \frac{r_n}{S_{n-1} - S_n} \quad 4.44$$





Algebraic Solution to the above equation

Divide $\frac{Q}{A}(S_o - S_e) = kS$ by $\frac{Q}{A}$ and S

$$\frac{S_o - S_e}{S} = \frac{k}{Q/A} \quad 4.45$$

By rearranging, the above equation gives the following

$$\frac{S}{S_o} = \frac{1}{1 + \frac{k}{Q/A}} \quad 4.46$$

For the first stage,

$$S_1 = \left(\frac{1}{1+\frac{k}{Q/A}}\right)S_o$$

For the second stage,

$$S_2 = \left(\frac{1}{1+\frac{k}{Q/A}}\right)S_1 = \left(\frac{1}{1+\frac{k}{Q/A}}\right) * \left(\frac{1}{1+\frac{k}{Q/A}}\right)S_o$$

Therefore,

$$\frac{S_2}{S_o} = \left(\frac{1}{1+\frac{k}{Q/A}}\right)^2$$

$$\frac{S_n}{S_o} = \left(\frac{1}{1+\frac{k}{Q/A}}\right)^n \qquad 4.47$$

The above equation represents the ***performance of RBC***.

Example:

A municipality with a design population of 6900 persons is to have a rotary biological contactor plant designed. Pertinent data are average flow = 100gal/cap/day (380lit/cap/day), influent BOD_5 = 200mg/l, primary clarifier removes 33% BOD_5, total effluent BOD_5 = 20mg/l, and number of stages = 4. If a pilot plant study gave a k value of 1.16gal/day-ft^2 (47.3l/d-m^2).

Determine:

i. The design hydraulic loading
ii. The BOD_5 after each stage
iii. The disk area per stage
iv. The total disk area

4.4.4 Waste Stabilization Pond

1. Introduction

Mara (1976) describes WSP as large shallow basins enclosed by earthen embankments in which wastewater is biologically treated by natural processes involving pond algae and bacteria. WSP comprise a single series of anaerobic, facultative and maturation ponds or several of such series in parallel. A long hydraulic retention time is necessary because of the slow rate at which the organic waste is oxidized. Typical hydraulic retention times range from 10 days to 100 days depending on the temperature of a particular region.

WSP are considered as the most effective and appropriate method of wastewater treatment in warm climates where sufficient land is available and where the temperature is most favorable for their operation. WSP are employed for treatment of a range of wastewaters, from domestic wastewater to complex industrial wastes. The design of WSP depends on the treatment objectives. It may be designed to receive untreated domestic or industrial wastes, to treat primary or secondary treatment plant effluents, excess activated sludge.

Anaerobic, facultative and maturation ponds are the three major types of pond in a WSP system. These ponds are normally arranged in series to achieve effective treatment of raw wastewater. Anaerobic and facultative ponds are employed for BOD removal, while maturation ponds remove excreted pathogens.

A series of anaerobic and facultative ponds can treat wastewater to a sufficient degree to allow it to be used in a restricted way for irrigating crops. It has been argued that such pond systems remove nematode eggs significantly by sedimentation (WHO, 1989). Maturation ponds are normally used if the treated wastewater is to be used for unrestricted crop irrigation complying with WHO guidelines of less than 1000 faecal coliforms (FC) per 100ml (WHO, 1989). Maturation ponds have also been used when stronger wastewaters with high concentrations of nutrients (nitrogen, phosphorus) are to be treated prior to surface discharge.

2. Advantages and Disadvantages of Waste Stabilization Ponds

Waste stabilization ponds (WSP) are shallow man-made basins into which wastewater flows and from which, after a retention time of several days (rather than several hours in conventional treatment processes), a well-treated effluent discharged. The advantages of WSP systems, which can be summarized; as simplicity, low cost and high efficiency, are as follows:

a. Simplicity

WSP are simple to construct: earthmoving is the principal activity; other civil works are minimal – preliminary treatment, Inlets and outlets, pond embankment protection and, if necessary, lining pond. They are also simple to operate and maintain: routine tasks comprise cutting the embankment grass, removing scum and any floating vegetation from the pond surface, keeping the inlets and outlets clear, and repairing any damage to the embankments, only unskilled, but carefully supervised, labor needed for pond O&M.

b. Low cost

Because of their simplicity, WSP are much cheaper than other is wastewater treatment processes. There is no need for expensive, electromechanical equipment (which requires regular skilled maintenance), nor for a high annual consumption of electrical energy.

c. High Efficiency

BOD removals > 90% readily obtained in a series of well-designed ponds. The removal of suspended solids is less, due To the presence of algae in the final effluent. Total nitrogen removal is 70 – 90%, and total phosphorus removals 30 – 45%. WSP are particularly efficient in removing excreted pathogens, whereas in contrast all other treatment processes are very inefficient in this, and require a tertiary treatment process such as chlorination (with all its inherent operational and environmental problems) to achieve the destruction of fecal bacteria.
A general comparison b/n WSP and conventional treatment processes for the removal of excreted pathogens is shown in the table below

Table1.2 Removals of excreted pathogens achieved by waste stabilization ponds and conventional treatment processes

Excreted pathogens	Removal in WSP	Removal in conventional treatment
Bacteria	Up to 6log unites [a]	1 - 2log units
Viruses	Up to 4 log unites	1 - 2log units
Protozoan cysts	100%	90 - 99%
Helminth eggs	100%	90 - 99%

[a]1log unit = 90%removal; 2 = 99%; 3 = 99.9% and so on

Sources: feachem et al.1983

3. Disadvantages of Waste Stabilization Ponds

The major disadvantage of WSPs is the large area that is required (2 - 5m^2/capita), the potentially high algal content of the effluent, evaporation losses, the potential odour and mosquito nuisance and the sensitivity of algae to toxic matter present in raw municipal sewage.

4. Types of Pond

1. Anaerobic Ponds

Anaerobic ponds are unmixed basins designed to enhance the settling and biodegradation of particulate organic solids by anaerobic digestion. Pond depth is usually between 3 to 5 meters and the HRT for ponds treating municipal sewage is between 1 - 3 days. For industrial applications, HRT may increase to 20 days. In cold climates, anaerobic ponds mainly act as settling ponds, whereas higher sewage temperatures enhance the anaerobic degradation process (hydrolysis, acidogenesis, acetogenesis and methanogenesis). At higher temperatures BOD is therefore more effectively removed, especially the BOD-dissolved. Typical TSS removal percentages range between 50 and 70%. BOD removal rate is increase with temperature and range between 30 and 75%.

i. Treatment Mechanisms

BOD removal is the combined effect of sedimentation and biological degradation. Biological degradation is due to the anaerobic degradation of complex organic material. In case the influent contains sulphate or nitrate, also sulphate reduction and denitrification is occurring. Through these latter two processes bacteria sustain their growth by using chemically bound oxygen to oxidize organic matter. However, the main process occurring in anaerobic ponds is the anaerobic degradation process. After hydrolysis of particulate organic matter, fermenting bacteria convert the readily biodegradable organic substrate into volatile fatty acids (VFAs). Higher VFAs are further decomposed, mainly into acetic acid and H_2, the typical substrate for the strict anaerobic methanogens. Effective anaerobic pond management has to avoid VFA accumulation and the associated drop in pH as methanogens are very sensitive to pH values less than 4 - 5. This problem is only encountered in anaerobic ponds treating concentrated industrial wastewaters, since the high concentration of organic material may lead to rapid VFA production and accumulation. The buffering capacity of industrial wastewater may not be sufficient to keep the pH in the desired range. In anaerobic ponds treating municipal sewage the pH is not a critical parameter since the buffering capacity of municipal sewage is sufficient to stabilize the pH.

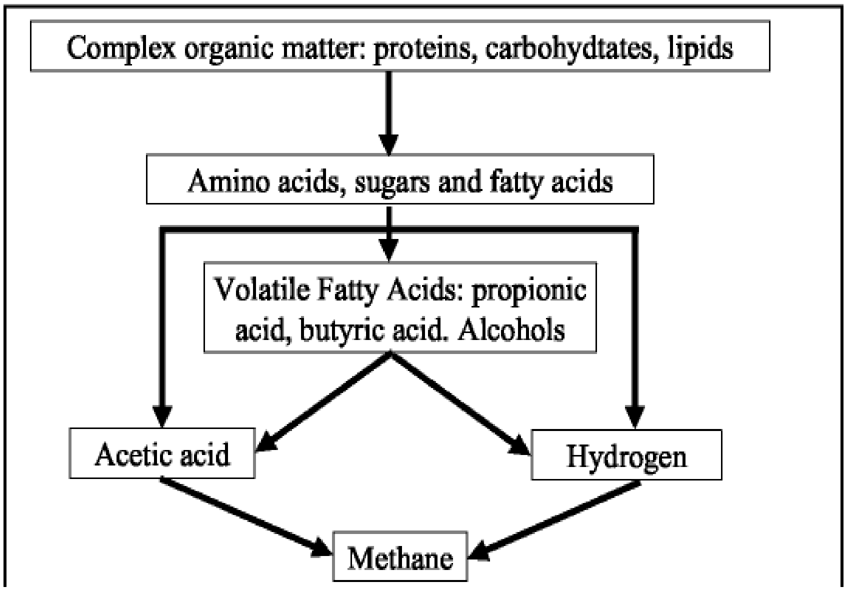


Figure 4-20: Treatment mechanisms in anaerobic pond

ii. Preliminary treatment

Anaerobic ponds require some preliminary treatment of municipal sewage. Usually coarse screening is applied to remove large pieces of wood, plastic etc. that could clog pipes or channels connecting the various ponds. In addition grit should be removed in a grit channel to prevent accumulation of the grit in the anaerobic pond that would reduce the active pond volume and increase the required de sledging frequency.

A) Screening

The influent is best split minimum into two influent Channels with a course screen. And it is recommended to add a fine screen to prevent the accumulation of unsightly plastic, on the pond surface. The screens are to be raked manually, and should be enough to prevent frequent clogging of the screens.

B) Grit Channels

Two Grit channels are recommended. When one channel is in operation, the other one can be manually, emptied.

C) Sludge Drying Beds

The size of sludge drying bed is based on the accumulation of sludge produced per year, i.e. (sludge accumulation rate x population equivalent). Commonly sludge beds are loaded with 30cm of sludge per drying cycle.

2. Facultative Ponds

Facultative ponds are the second treatment step in a pond system. In facultative ponds the anaerobic pond effluent is further treated, aimed at further BOD, nutrient and pathogen removal. Facultative ponds are usually 1.5 - 2.5m deep. The HRT for ponds treating anaerobic effluent varies between 5 and 30 days. Facultative ponds are most widely used for treatment of municipal wastewater following aerated or anaerobic ponds. Filtered effluent BOD values range from 20 to 60mg/l, while TSS levels vary from 30 to 150mg/l. Facultative ponds are normally followed by maturation ponds to further polish its effluent.

i. Processes

The aquatic environment of facultative ponds is a complicated ecosystem where a large number of interacting processes are occurring simultaneously. In facultative ponds the waste stabilization is the result of both oxidation of organic matter by aerobic and facultative bacteria as well as anaerobic processes in the anaerobic bottom layer. The name 'facultative' is actually derived from the fact that the top layer of facultative ponds is aerobic due to oxygen production by algae and the bottom layer is anaerobic due to the absence of algae activity. The management principle of facultative ponds is to balance the oxygen input by photosynthetic algae and surface re aeration to the oxygen demand exerted by organic matter oxidation. The basic symbiosis underlying the concept of facultative ponds is that the oxygen produced by algal photosynthesis in the top layer is used for the decomposition of organic matter in deeper layers by heterotrophy. This symbiotic interrelationship is referred to as 'Algae-Bacteria Symbiosis'.

To sustain algae growth and photosynthesis the supply of macro-nutrients (N.P.K) is essential. A BOD/N/P ratio of 100/5/1 is generally recommended to satisfy the basic needs.

ii. BOD Removal Mechanisms

The consumption of oxygen produced by photosynthesis and re aeration is mainly due to bacterial oxidation of BOD and ammonium. The two main mechanisms for BOD removal not involving oxygen are sedimentation and anaerobic digestion. Sedimentation results only in temporary storage of BOD in the sludge layer. This BOD is removed while the pond is desludged. Part of the sludge BOD is however anaerobic ally transformed into methane gas. The methane gas and its associated BOD leave the pond system via escaping gas bubbles or by diffusion.

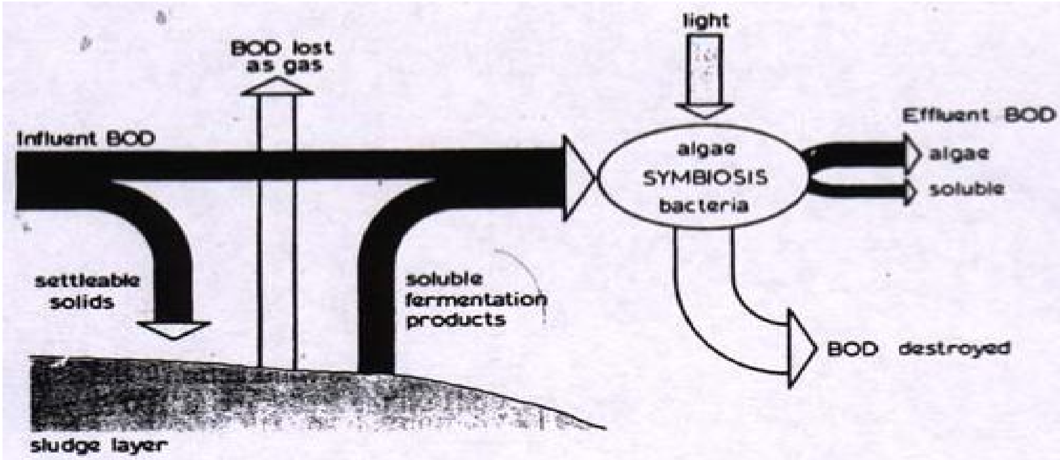


Figure 4-21: BOD removal mechanisms in a facultative pond

3. Maturation Ponds

Maturation ponds are shallow ponds in which an active algal biomass is maintained throughout the entire depth of the system so that during daytime large amounts of oxygen are produced. Further stabilization of organic matter and nutrient removal accomplished mainly through aerobic bacteria, while pathogen, destruction realized via a complex interaction of various environmental factors.

Maturation ponds are entirely aerobic and 1 - 1.5m deep. Faecal coliform and virus die-off rates may reach over 3 to 4 log units. Cysts and ova of intestinal parasites are more resistant but as they have relative densities higher than one they may effectively settle to the pond bottom where they eventually die-off.

BOD removal in maturation ponds is much slower than in facultative ponds, since the most easily degradable substances consumed already. In addition, experimental results showed no correlation between BOD removals in maturation ponds with temperature or retention time (Mara et al., 1992). For design purposes, it recommended to assume 25% BOD removal (based on BOD influent-total and BOD effluent-soluble) in maturation ponds (Mara and Pearson, 1992). The high amount of algal biomass in the effluent represents a high-suspended matter concentration, which may exceed the final effluent quality guidelines. Typically the oxygen demand exerted by these suspended algal material is around 0.5 - 0.6mg BOD_5/mg algal TSS.

The major application for maturation ponds is to polish or upgrade facultative pond effluents and achieve substantial microbial reductions to allow safe use of the effluents in agriculture or aquaculture.

i. Removal of Pathogenic Microorganisms

Pathogen removal occurs in anaerobic, facultative and maturation ponds, but only maturation ponds are designed on the basis of required removal rates for pathogens. It is in maturation ponds that the environmental conditions are most harmful for pathogens. Pathogens present in municipal sewage four groups of pathogenic micro-organism can be distinguished: bacteria, viruses, protozoa and helminthes. Faecal coliforms are a group of bacteria that is commonly used as an indicator for contamination with faecal material. Contamination with faecal material means potentially contamination with pathogenic bacteria like Salmonella, Shigella or Vibrio cholerae. The non-pathogenic group of faecal coliforms (FC) is used as an indicator because of practical and safety reasons.

Both helminth eggs and protozoan cysts are removed by sedimentation. Their removal is therefore mostly affected by retention time. Waste stabilizations pond systems with total HRT of 15 - 62 days. Concentrations in the influent were up to 73 and 6200 cysts/liter for Giardia and Cryptosporidium respectively.

It is not possible due to health hazards to analyze for bacterial pathogens in standard monitoring program. Therefore so-called indicator organisms are used, of which the most common is the group of faecal coliforms (FC), itself not pathogenic. One characteristic of this indicator is that it is just as persistent as or more persistent than the real pathogens. So, if faecal coliforms have been removed completely, it is safe to assume that bacterial pathogens are absent.

A distinction can be made between FC removal and decay (decay is sometimes called die-off). Removal is defined as the reduction of the faecal coliform count in the pond effluent as compared to the influent. FC that has been removed not necessarily has decayed, since they may be attached to solids that settled to the sediment. Removal is due to a combination of several processes:

- Adsorption to particles and subsequent sedimentation
- Grazing by other micro-organisms (protozoa)
- Natural decay

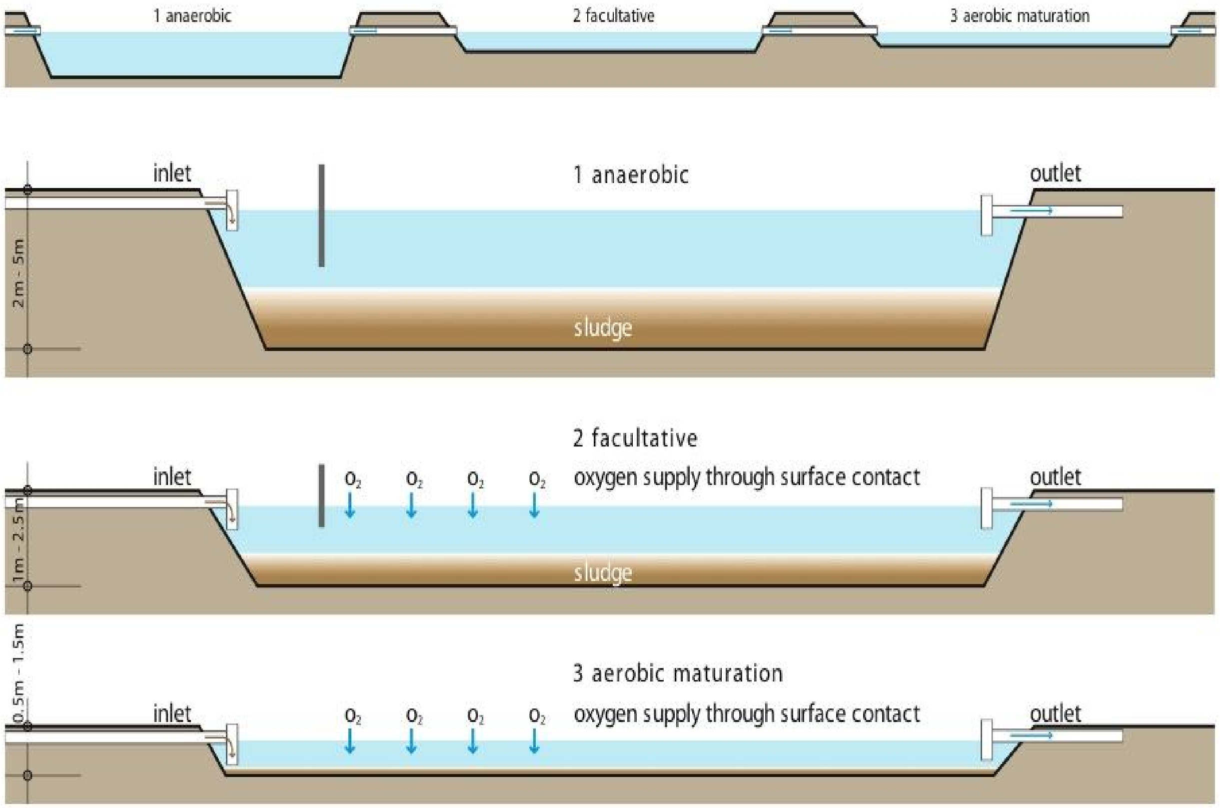


Figure 4-22: Typical scheme of a waste stabilization system

5. Physical Design of WSP

i. Pond Location

Ponds should be located at least 200m (preferably 500m) downwind from the community they serve and away from any Likely area of future expansion. Odour release, even from anaerobic ponds, is most unlikely to be a problem in a well-designed and properly maintained system, but the public May need assurance about this at the planning stage, and a minimum distance of 200 m normally allays any fears. There should be vehicular access to the ponds and, so as to minimize earthworks, the site should be flat or gently sloping. The Soil must also be suitable. Ponds should not be Located within 2km of airports, as any birds attracted to the ponds may constitute a risk to air navigation.

To facilitate wind-induced mixing of the pond surface layers, the ponds should be located so that its length lies in the direction of the prevailing wind direction. In particular in summer time when stratification is at its greatest this wind induced mixing has to be optimal.

ii. Preliminary Treatment

Adequate screening and grit removal facilities must be installed at all but very small systems (those serving < 1000 people). Adequate provision must be made for the hygienic disposal of screenings and grit; haulage to a sanitary landfill or onsite burial in trenches is usually the most appropriate method.

iii. Pond Geometry

There has been little rigorous work done on determining optimal pond shapes. The most common shape is rectangular, although there is much variation in the length-to-breadth ratio. Clearly, the optimal pond geometry, which includes not only the shape of the pond but also the relative positions of its inlet and outlet, is that which minimizes hydraulic short-circuiting. In general, anaerobic and primary facultative ponds should be rectangular, with length-to-breadth ratios of 2 – 3 to 1 so as to avoid sludge banks forming near the inlet. Secondary facultative and maturation ponds should, wherever possible, have higher length-to-breadth ratios (up to 10 to 1) so that they better approximate plug flow conditions. Ponds do not need to be strictly rectangular, but may be gently curved if necessary or if desired for aesthetic reasons. A single inlet and outlet are usually sufficient, and these should be located just away from the base of the embankment in diagonally opposite corners of the pond (the inlet should not discharge centrally in the pond as this maximizes hydraulic short-circuiting). The use of complicated multi-inlet and multi-outlet designs is unnecessary and not recommended. To facilitate wind-induced mixing of the pond surface layers, the pond should be located so that its longest dimension (diagonal) lies in the direction of the prevailing wind. If this is seasonally variable, the wind direction in the hot season should be used, as this is when thermal stratification is at its greatest. To minimizes hydraulic short-circuiting, the inlet should be located such that the wastewater flows in the pond against the wind. The minimum freeboard that should be provided is decided on the basis of preventing waves, induced by the wind, from overtopping the embankment. For small ponds (under 1 ha in area) 0.5m freeboard should be provided; for ponds between 1 ha and 3 ha, the freeboard should be 0.5 - 1m, depending on site considerations.

iv. Pond Configurations

Configurations can includes either series or parallel operations the advantages of series operation is improved treatment because of reduced short circuiting and the advantages of parallel configuration is that the loading can be distributed more uniformly over a large area combinations of parallel & series operation can be accomplished along with recalculation.

v. Inlet and Outlet Structures

There are a wide variety of designs for inlet and outlet structures, and provided they follow certain basic concepts, their precise design is relatively unimportant. Firstly, they should be simple and inexpensive; while this should be self-evident, it is all too common to see unnecessarily complex and expensive structures. Secondly, they should permit samples of the pond effluent to be taken with ease. The inlet to anaerobic and primary facultative ponds should discharge well below the liquid level so as to minimize short-circuiting (especially in deep anaerobic ponds) and thus reduce the quantity of scum (which is important in facultative ponds). Inlets to secondary facultative and maturation ponds should also discharge below the liquid level, preferably at mid-depth in order to reduce the possibility of short-circuiting. The outlet of all ponds should be protected against the discharge of scum by the provision of a scum guard. The take-off level for the effluent, which is controlled by the scum guard depth, is important as it has a significant influence on effluent quality. In facultative ponds, the scum guard should extend just below the maximum depth of the algal band when the pond is stratified so as to minimize the daily quantity of algae, and hence BOD, leaving the pond.

6. Process Design of WSP

i. Effluent Quality Requirements

The general WHO guideline standards for the discharge of treated wastewaters into inland surface waters are given in the environmental (protection) Rules. The more important of these for WSP design are as follows:

Parameter	**Effluent limit**
BOD	30 mg/l
Suspended solids	100 mg/l
Total N	100 mg N/l
Total ammonia	50 mg N/l
Free ammonia	5 mg N/l
Sulphide	2mg/l
pH	5.5 – 9.0

ii. Design Parameters

The four most important parameters for WSP design are temperature, net evaporation, flow and BOD. Faecal coliform and helminth egg numbers are also important if the final effluent is to be used in agriculture or aquaculture.

iii. Temperature and Net Evaporation

The usual design temperature is the mean air temperature in the coolest month (or quarter). Another design temperature commonly used is the air temperature in the coolest period of the irrigation season. Net evaporation has to be taken into account in the design of facultative and maturation ponds, but not in that of anaerobic ponds, as these generally have a scum layer which effective prevents significant evaporation. The net evaporation rates in the months used for selection of the design temperatures are used; additionally a hydraulic balance should be done for the hottest month

iv. Flow

The mean daily flow should be measured. It must be estimated very carefully since the size of the ponds, and hence their cost, is directly proportional to the flow. The wastewater flow should not be based on the design water consumption per caput, as this is unduly high since it contains an allowance for losses in the distribution system. A suitable design value is 80 percent of the in-house water consumption, and this can be readily determined from records of water meter readings. If these do not exist, the actual average 24-hour wastewater flow from outfall drains can be measured; or alternatively the design flow may be based on local experience in sewered communities of similar socio-economic status and water use practice.

v. BOD

The BOD may be measured using 24-hour flow-weighted composite samples. If wastewater does not yet exist, it should be estimated from the following equation:

$$C_i = \frac{1000B}{q} \tag{4.48}$$

Where C_i = wastewater BOD concentration, mg/l

B = BOD contribution, g/c/d

q = wastewater flow, l/c/d

Values of B vary between 30 and 70g/c/d, with affluent communities producing more BOD than poor communities. As suitable design value for Ethiopia is 45g per caput per day (source; Adopted from Addis Ababa Water Sewerage Authority report 2003)

vi. Faecal Coliforms

Faecal coliform numbers are important if the pond effluent is to be used for unrestricted crop irrigation or for fishpond fertilization. Grab samples of the wastewater may be used to measure the faecal coliform concentration if the wastewater exists. The usual range is 10^7- 10^8 faecal coliforms per 100 ml, and a suitable design value is $5*10^7$ per 100 ml.

vii. Helminth Eggs

Helminth egg numbers are also important when pond effluents are used for restricted crop irrigation (irrigation of all crops except salads and vegetables eaten uncooked) or fishpond fertilization. Composite samples may be used to count the number of human intestinal nematodes eggs. The usual range is 100 - 1000 eggs per liter.

viii. Loading and Retention Time

Any pond treatment system requires steady effluent flow to encourage the rapid and continuous growth of bacteria involved in the biological breakdown of effluent it is essential that the daily loading into the ponds be kept to the design standards of the pond system. A very large load may flush out important bacteria, eventually leading to system failure. Variation in loads will alter the retention time. Any attempt to extend the time that effluent remains within the pond system will increase the amount of disease-causing microorganism die-off. The concentration of microorganisms within the effluent will be reduced and the effluent will be of higher quality before discharge into a waterway.

7. Operation and Maintenance of WSPs

i. WSP start-up

Before commissioning a WSP system, any vegetation growing in the empty ponds must be removed. The facultative ponds and maturation ponds are commissioned before the anaerobic ponds so as to avoid odour release when the anaerobic pond effluent discharges into empty facultative ponds. The facultative ponds and maturation ponds should ideally be filled initially with fresh surface water or groundwater to permit the development of the required algal and heterotrophic bacterial populations. If freshwater isn't available, then the facultative pond can be filled with raw wastewater and allowed to rest in batch mode for 3−4 weeks to allow the microbial populations to develop. Some odour release may be expected during this period. Once the facultative ponds and maturation ponds have been commissioned, the anaerobic ponds are filled with raw wastewater and, if possible, inoculated with active biomass (sludge seed) from another anaerobic bioreactor. The anaerobic ponds are then loaded gradually up to their design load over a period of

2 − 4 weeks (the time depends on whether the anaerobic pond was inoculated with an active sludge seed or not). The pH of the anaerobic pond has to be maintained at around 7 − 7.5 during the start-up to allow for the methanogenic archaeal populations to develop. If the pH falls below 7 during this period, lime should be added to correct it.

ii. Routine Maintenance

Once the ponds have started functioning in steady state, routine maintenance is minimal but essential for good operation. The main routine maintenance activities are:

- Removal of screenings and grit from the preliminary treatment units
- Periodically cutting the grass on the pond embankments
- Removal of scum and floating macrophytes from the surface of facultative ponds and maturation ponds. This is done to maximize the light energy reaching the pond algae, increase surface re-aeration, and prevent fly and mosquito breeding
- If flies are breeding in large numbers on the scum on anaerobic ponds, the scum should be broken up and sunk with a water jet
- Removal of any material blocking the pond inlets and outlets
- Repair of any damage to the embankments caused by rodents or rabbits (or other burrowing animals)
- Repair of any damage to fences and gates

iii. Desludging and Sludge Disposal

Anaerobic ponds required desludged when they are one third full of sludge (by volume).this occurred every n years where n is given by

$$n = \frac{v}{3ps} \qquad 4.49$$

Where v = volume of anaerobic pond, m^3

p = population served

s = sludge accumulation rate, m^3/capita year

8. Process Design of WSP

a. Anaerobic Ponds

Designed without risk of odour & nuisance on the basis of volumetric BOD loading (λv, $g/m^3/d$), which is given by:

$$\lambda v = \frac{C_i Q}{V} \qquad 4.50$$

Where C_i = influent BOD concentration, mg/l

Q = flow, m^3/d, V = anaerobic pond volume, m^3

The permissible design value of λv increases with temperature, but there are too few reliable data to permit the development of a suitable design equation. (Mara and Pearson, 1986 and Mara et al., 1997) recommend the design values given in Table 3.1 which may be safely used for design purposes in Ethiopia. These recommendations were based on those of Meiring et al. (1968) that λV should lie between 100 and 400g/m^3/d, the former in order to maintain anaerobic conditions and the latter to avoid odour release (see also Mara and Mills, 1994).

Volumetric loading rate λ_v: linear increase (Arthur 1983)

At 12^0c, $\lambda_v = 0.1 kg\ BOD/m^3/day$ to maintain anaerobic conditions

At 30^0c, $\lambda_v = 0.4 kg\ BOD/m^3/day$ to avoid odour release

Table 2.1 Volumetric loading (g/m^3/d)

Temperature (^{0}C)	Volumetric loading (g/m^3d)	BOD removal (%)
< 10	100	40
10 - 20	20T - 100	2T + 20
20 - 25	10T + 100	2T + 20
> 25	350	70

Source: Mara and Pearson, 1986 and Mara et al., 1997

Once a value of V has been selected, the anaerobic pond volume is then calculated from equation 2.2. The mean hydraulic retention time in the pond (θ, d) is determined from:

$$\theta = \frac{V}{Q} \qquad 4.51$$

Retention times in anaerobic ponds < 1 day should not be used. If equation 2.3 gives a value of θ < 1 day, a value of 1 day should be used and the corresponding value of V recalculated from equation 3.2.

b. Facultative Ponds

Although there are several methods available for designing facultative ponds (Mara, 1976), it is recommended that they be designed on the basis of surface BOD loading(λS, kg/ha/d), which is given by:

$$\lambda s = \frac{10 C_i Q}{A_f} \qquad 4.52$$

Where A_f = facultative pond area, m^2

The variation of permissible design value for λS with latitude is given by (Arceivala et al., 1970). This relationship can be expressed mathematically as:

$$\lambda s = 375 - 6.25L \qquad 4.53$$

Where L = latitude,

Here the permissible design value of λS increases with temperature (T, ^{0}C). The earliest relationship between λS and T is that given by McGarry and Pescod (1970), but their value of λS is the maximum that can be applied to a facultative pond before it fails (that is, becomes anaerobic). Their relationship, which is therefore an envelope of failure, is:

$$\lambda s = 60(1.099)^T \qquad 4.54$$

However, a more appropriate global design equation was given

By Mara (1987):

$$\lambda s = 350(1.107 - 0.002T)^{T-25} \qquad 4.54$$

Once a suitable value of λS has been selected, the pond area is calculated from equation (2.4) and its retention time (θ_f, d) from:

$$\theta_f = \frac{A_f D}{Q_m}$$

Where: D = pond depth, m (usually 1.5m), Q_m = mean flow, m^3/day

The mean flow is the mean of the influent and effluent flows (Q_i and Q_e), the latter being the former less net evaporation and seepage. Thus equation 2.8 becomes:

$$\theta_f = \frac{A_f D}{\frac{1}{2}(Q_i + Q_e)} \qquad 4.55$$

If seepage is negligible, Q_e is given by:

$$Q_f = Q_i - 0.001 A_f e$$

Where e = net evaporation rate, mm/day. Thus equation 2.9 becomes:

$$\theta_f = \frac{2 A_f D}{(2Q_i - 0.001 A_f e)} \qquad 4.56$$

A minimum value of θ_f of 5 days should be adopted for temperatures below 20^{o}C, and 4 days for temperatures above 20°C

c. **Maturation Ponds**

i. **Faecal Coliform Removal**

The method of Marais (1974) is generally used to design a pond series for faecal coliform removal. This assumes that faecal coliform removal can be modeled by first order kinetics in a completely mixed reactor. The resulting equation for a single pond is thus:

$$N_e = N_i(1 + k_T * HRT) \quad 4.57$$

Where N_e = number of FC per 100 ml of effluent

N_i = number of FC per 100 ml of influent

k_T = first order rate constant for FC removal, d^{-1}

HRT = retention time, d

For a series of anaerobic, facultative and maturation ponds,

Equation (4.57) becomes:

For a series of anaerobic, facultative and maturation ponds, the above equation becomes:

$$N_e = \frac{N_i}{(1 + k_T * HRT_{anearobic}) * (1 + k_T * HRT_{facultative}) * (1 + k_T * HRT_{maturation,n})^n} \quad 4.58$$

Where:

N_e = number of feacal coliform per 100 ml effluent

N_i = number of feacal coliform per 100 ml influent

k_T = first order temperature dependent rate (day^{-1})

n = number of maturation ponds (each pond the same hydraulic retention rate)

A series of n maturation ponds should have total HRT of 5 days.

The value of kT is highly temperature dependent. (Arthur, 1983)

Found that:

$$k_T = 2.6(1.19)^{T-20} \quad 4.59$$

Thus k_T changes by 19% for every change in temperature of 1 $^{\circ}$C (Table 4-5).

Table 4-5: Values of the k_T for faecal coliform removal at various temperatures

T(^{0}C)	**k_T(day^{-1})**	**T(^{0}C)**	**k_T(day^{-1})**
11	0.54	21	3.09
12	0.65	22	3.68
13	0.77	23	4.38
14	0.92	24	5.21
15	0.09	25	6.20
16	1.30	26	7.38
17	1.74	27	8.77
18	1.84	28	10.47
19	2.18	29	12.44
20	2.6	30	14.6

Source :(Arthur, 1983)

ii. Helminth Egg Removal

Analysis of egg removal data from ponds in Brazil, India and Kenya (Ayres et al.1992) has yielded the following relationship which is equally valid for anaerobic, facultative and maturation ponds:

$$R = 100\,[1 - 0.14\exp(-0.38\theta)] \qquad 4.60$$

Where R = percentage egg removal

θ = retention time, d

The equation corresponding to the lower 95% confidence limit of equation (4.60) is:

$$R = 100\left[1 - 0.41\exp(-0.49\theta + 0.0085\theta_2)\right] \qquad 4.61$$

Table 4-6: Design values of helminth egg removal (R %) for hydraulic retention times (θ)

θ	R	θ	R	θ	R	θ	R
1.0	74.67	3.2	90.68	6.0	97.06.	12	99.61
1.2	76.95	3.4	91.45	6.5	97.57	13	99.70
1.4	79.01	3.6	92.16	7.0	97.99	14	99.77
1.6	80.87	3.8	92.80	7.5	98.32	15	99.82
1.8	82.55	4.0	93.38	8.0	98.60	16	99.86
2.0	84.08	4.2	93.66	8.5	98.82	17	99.88
2.2	85.46	4.4	93.40	9.0	99.01	18	99.90
2.4	87.72	4.6	94.85	9.5	99.16	19	99.92
2.6	87.85	4.8	95.25	10	99.29	20	99.93
2.8	88.89	5.0	95.62	10.5	99.39		
3.0	89.82	5.5	96.42	11	99.38		

Source: Ayres et al.(1992)

iii. Nutrient Removal

Pano and Middlebrooks (1982) present equations for ammonical nitrogen (NH_3^+ and NH_4^+) removal in individual facultative and maturation ponds. Their equation for temperatures below 20°C is:

$$C_e = \frac{C_i}{\{1 + [(A/Q)(0.0038 + 0.000134T)\exp((1.041 + 0.044T)(pH - 6.6))]\}}$$

And for temperatures above 20°C:

$$C_e = \frac{C_i}{\left\{1 + \left[5.035 * 10 - 3 * \frac{A}{Q}\right]\left[\exp(1.54 * (pH - 6.6))\right]\right\}} \qquad 4.62$$

Where:

C_e = ammonical nitrogen concentration in pond effluent (mg N/l)

C_i = ammonical nitrogen concentration in pond influent (mg N/l)

A = pond surface area (m^2)

Q = wastewater flow rate (m^3/day)

T = temperature (°C)

Reed (1985) presents an equation for the removal of total nitrogen in individual facultative and maturation ponds:

$$C_e = C_i \exp\{-[0.0064(1.039)^{T-20}]\,[\theta + 60.6(pH - 6.6)]\} \qquad 4.63$$

Where C_e = total nitrogen concentration in pond effluent, mg N/l

C_i = total nitrogen concentration in pond influent, mg N/l

T = temperature, °C (range: 1 - 28°C)

θ = HRT = hydraulic retention time, d (range 5 - 231 d)

The pH value used in equations 3.20 - 3.22 may be estimated from:

$$pH = 7.3\exp(0.0005A) \qquad 4.64$$

Where A = influent alkalinity, mg $CaCO_3$/l

Equations 3.20 - 3.22 are applied sequentially to individual facultative and maturation ponds in the series, so that concentrations in the effluent can be determined

iv. Phosphorus

There are no design equations for phosphorus removal in WSP. Huang and Gloyna (1984) indicate that, if BOD removal in a pond system in 90 percent, the removal of total phosphorus is around 45%. Effluent total P is around two-thirds inorganic and one third organic.

9. Hydraulic Balance

To maintain the liquid level in the ponds, the inflow must be at least greater than net evaporation and seepage at all times. Thus:

$$Q_i \geq 0.001A * (e + s) \qquad 4.65$$

Where Q_i = inflow to first pond, m^3/d

A = total area of pond series, m^2

e = net evaporation (i.e. evaporation less rainfall), mm/d

s = seepage, mm/d

Seepage losses must be at least smaller than the inflow less net evaporation so as to maintain the water level in the pond. The maximum permissible permeability of the soil layer making up the pond base can be determined from Darcy's law:

$$k = \frac{Q_s}{(86{,}400A)} \frac{\Delta l}{\Delta h} \qquad 4.66$$

Where k = maximum permissible permeability, m/s

Qs = maximum permissible seepage flow (= Q_i – 0.001Ae), $m^{3/}d$

A = base area of pond, m^2

Δl = depth of soil layer below pond base to aquifer or more permeable stratum, m

Δh = hydraulic head (= pond depth + Δl), m

If the permeability of the soil is more than the maximum permissible, the pond must be lined. A variety of lining materials is available and local costs dictate which should be used.

The following interpretations may be placed on values obtained for the in situ coefficient of permeability:

- $k > 10^{-6}$ m/s: the soil is too permeable and the ponds must be lined
- $k > 10^{-7}$m/s: some seepage may occur but not sufficiently to prevent the ponds from filling
- $k < 10^{-8}$ m/s: the ponds will seal naturally
- $k < 10^{-9}$ m/s: there is no risk of groundwater contamination
- (If $k > 10^{-9}$ m/s and the groundwater is used for potable supplies, further detailed hydrogeological studies may be required).

Design Example on Waste Stabilization Pond

Data:

- Site characteristics and conditions
- Town of 20,000 population
- Consumption of 150l/c/day and wastage of 85%
- No significant infiltration into sewer system
- Average BOD production of 45g BOD/c/day
- Measured influent concentration of $4*10^8$ FC/100ml
- Clay bottom (hydraulic conductivity 10^{-7}m/s)
- Climate of the area (Latitude = ± 16°S)

Maximum monthly temperature	33°c (September)
Minimum monthly temperature	27°c (June/July)
Total annual rainfall	1143mm
Maximum monthly rainfall	206mm (November)
Minimum monthly rainfall	15mm (August)

Requirements

- Sludge remove in anaerobic ponds only once every 2 years
- Design each pond with a freeboard of 0.5m
- The treated effluent must have a BOD concentration below 20mg/l and should be reusable for agricultural purposes (use standards according to WHO)

Task:

- Design a conventional WSP system (anaerobic + facultative + maturation ponds).
- Provide for each pond the dimensions (L, W and D), the volume, surface area and the residence time.
- Calculate C_{in} and C_{out} from each pond.

Preliminary calculations:

- Influent flow Q:

$$Q = 20{,}000 * 0.15m^3/c * 0.85 = 2550m^3/day$$

- Influent BOD concentration C_i:

$$C_i = \frac{45 * 10^3 \text{ mg BOD/c/day}}{150L/c/day * 0.85} = 353mg\ BOD/L$$

- Influent BOD-load, Li:

$$L_i = 0.045kgBOD/capita.day * 20{,}000 = 900kgBOD/day$$

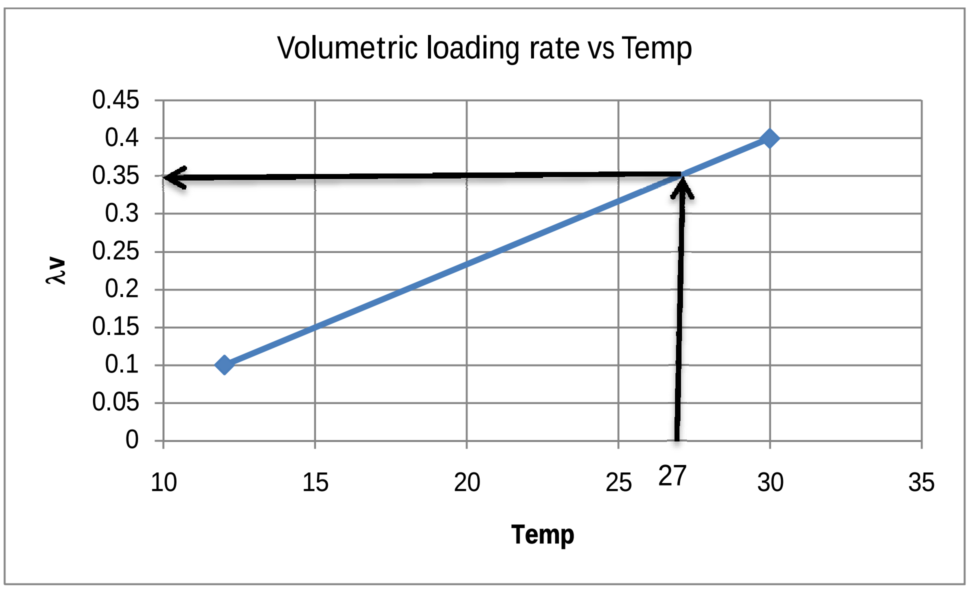
- Clay bottom with low hydraulic conductivity
  - Limited infiltration (10^{-7}m/s = 0.36mm/h = 8.64mm/day)
  - No lining is necessary
- Precipitation and evapo(transpi)ration will influence the system but since no detailed data are available this aspect will not be considered.
  - Precipitation causes dilution (concentration reduction) but also increases the flow rate and hence the hydraulic residence time decreases (efficiency reduces)
  - Evaporation causes increase in concentration but also decreases the flow rate and hence the HRT increases (efficiency increase).

Anaerobic pond

Calculations done at 27°c (coldest temperature) = worst case scenario

Arthur (1983):

Volumetric loading rate at 27°c = 0.35kg BOD /m^3/day



Required volume:

$$V_{wastewater} = \frac{900\text{kg BOD/day}}{0.35\text{kg BOD/m}^3\text{. day}} = 2571\text{m}^3$$

WHO (1997): 40 liter sludge per capita per year (sludge generation rate)

$$V_{sludge} = 0.04\text{m}^3\text{sludge/capita/year} * 20{,}000 * 2\text{years} = 1600\text{m}^3$$

Total volume:

$$= 2571 + 1600 = 4171\text{m}^3$$

Resulting hydraulic residence time (HRT):

$$= \frac{2571m^3}{2550m^3/day} = 1.01day\ \text{(after two years sludge accumulation)}$$

$$= \frac{4171m^3}{2550m^3/day} = 1.64\ day\ \text{(no sludge present in the pond)}$$

Resulting mid-depth area A for standard pond depth of 4m:

$$A = \frac{4171}{4} = 1043m^2$$

Pond shape: normally square (equal length and width)

Slope: usually 33% for stability reasons (0.33m vertical rise per 1m of horizontal progress)

Side of square at half depth

$$= \sqrt{area} = \sqrt{1043} = 32m$$

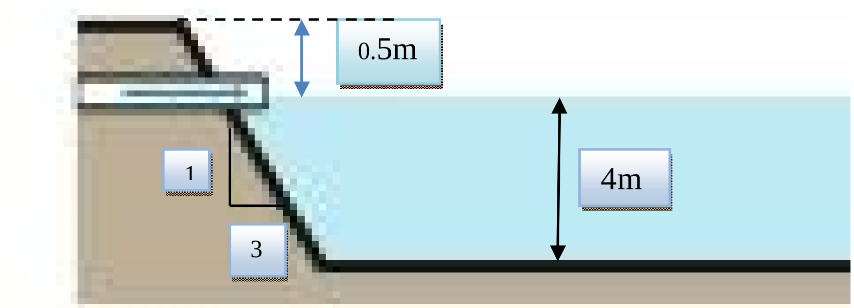
Side of square at the water surface

$$= \text{side at half depth} + (2 * (0.5 * depth) * \text{inverse slope})$$

$$= 32 + (2 * 2 * 3) = 44m \rightarrow area = 1936m^2$$

Side of square including 0.5m freeboard

$$= 44 + (2 * 0.5 * 3) = 47m \rightarrow area = 2209m^2$$



Expected BOD removed: **70%**

Effluent BOD concentration:

$$C_e = 353 - (0.7 * 353) = \mathbf{106} mg\ BOD/l$$

Effluent load:

$$L_e = 0.106\ kg\ BOD/m^3 * 2550m^3/day = \mathbf{270} kg\ BOD/day$$

Facultative Pond:

Calculations done at 27°c (coldest temperature) = worst case scenario

Design: based on surface loading rate λs (BOD/ha/day)

- Arthur (1983): $\lambda s = (20 * T) - 60 = (20 * 27) - 60 = 480kg\ BOD/ha/day$
- Arceivala: $\lambda s = 375 - (6.25 * L) = 375 - (6.25 * 16) = 275kg\ BOD/ha/day$
- McGarry and Pescod: $\lambda s = 60 * 1.099^T = 777kg\ BOD/ha/day$

- Mara: $\lambda s = (20 * T) - 120 = (20 * 27) - 120 = 420\text{kg BOD/ha/day}$

Take 500kg BOD/ha/day (slightly above average value of 488)

Mid-depth area

$$= \frac{270\text{kg BOD/day}}{500\text{kg BOD/ha/day}} = 0.54\text{ha} = 5400\text{m}^2$$

Resulting volume if standard pond depth of 1.8m

$$V = A * D = 5400 * 1.8 = 9720\text{m}^3$$

$$HRT = \frac{V}{Q} = \frac{9720}{2550} = 3.8 \text{ days}$$

Usually rectangular pond with aspect ratio 2:1 (L = 2 * W), slope 33% is adopted.

$$5400\text{m}^2 = L * W = (2 * W) * W = 2W^2$$

$$W = 52\text{m and } L = 104\text{m (at half depth)}$$

$$W = 57.5\text{m and } L = 109.5\text{m (at water surface)}$$

$$W = 60.5\text{m and } L = 112.5\text{m (including free board)}$$

Expected BOD removal: 80%

Effluent BOD concentration: 106 – (0.8*106) = **21.2**mg/ BOD/l

Effluent load: 0.0212kg BOD/m^3 * 2550 m^3/day = **54.1**kg BOD/day

Maturation pond:

Calculations done at 27°c (coldest temperature) = worst case scenario

$$N_e = \frac{N_i}{(1 + k_T * HRT_{anaerobic}) * (1 + k_T * HRT_{facultative}) * (1 + k_T * HRT_{maturation,n})^n}$$

Where:

N_e = 1000 FC/100ml (required by WHO for agricultural reuse)

N_i = 4*10^8 FC/100ml (given concentration)

$$k_T = k_{20°c} * \theta^{(T-20)} = 2.6 * (1.19)^{27-20} = 8.8\ \text{day}^{-1} \quad \text{(Arthur, 1983)}$$

$$k_T = k_{20°c} * \theta^{(T-20)} = 2.0 * (1.07)^{27-20} = 3.2\ \text{day}^{-1} \quad \text{(WHO, 1987)}$$

HRT anaerobic = 1.01 day (worse case, if full of sludge)

HRT facultative = 3.8 days

HRT maturation =?

Number of maturation ponds =?

Determine via trial and error procedure

- Arthur: a series of maturation ponds with a total HRT of 5 days
- WHO: 1 maturation pond of 5 days or several maturation ponds of 3 days

Calculate N_e for different numbers of maturation ponds in series and check whether or not N_e is below the standard.

n	HRT per pond	N_e (Arthur)	HRT per pond	N_e (WHO)
1	5	26,102 FC/100ml	5	422,483 FC/100ml
2	2.5	2,220 FC/100ml	3	63,921 FC/100ml
3	1.67	304 FC/100ml	3	6,030 FC/100ml

Retain Arthur solution, select three ponds, each with a HRT of 1.67 days.

Volume per pond:

$$V = 1.67\text{ days} * 2550\text{m}^3/\text{day} = 4258\text{m}^3$$

Take standard depth of 1.5m

Mid depth area

$$A = \frac{V}{D} = \frac{4258}{1.5} = 2839\text{m}^2$$

Normally L:W = 2:1

W = 38m and L = 76m (at half depth)

W = 42.5m and L = 80.5m (at water surface)

W = 45.5m and L = 83.5m (including free board)

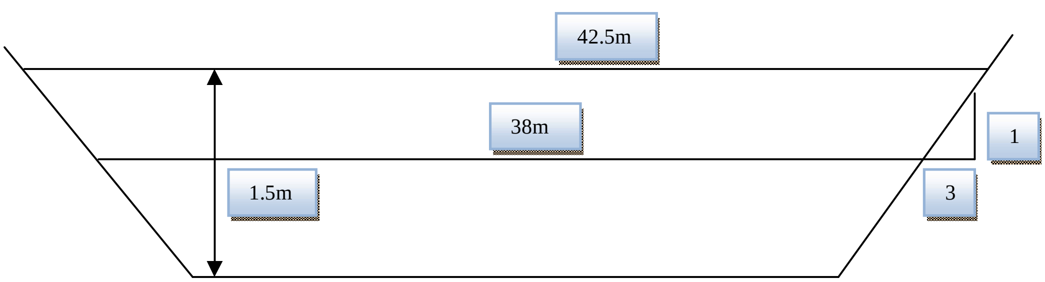


Figure 4-23: Sectional view of maturation pond

4.4.5 Constructed Wetlands

1. General

Constructed wetlands (CWs) are planned systems designed and constructed to employ wetland vegetation to assist in treating wastewater in a more controlled environment than occurs in natural wetlands. Hammer (1990) defines constructed wetlands as a designed, manmade complex of saturated substrate, emergent and submerged vegetation, animal life, and water that simulate wetlands for human uses and benefits. Constructed wetlands are an "eco-friendly" alternative for secondary and tertiary municipal and industrial wastewater treatment. The pollutants removed by CW's include organic materials, suspended solids, nutrients, pathogens, heavy metals and other toxic or hazardous pollutants. In municipal applications, they can follow traditional sewage treatment processes. Different types of constructed wetlands can effectively treat primary, secondary or tertiary treated sewage. However wetlands should not be used to treat raw sewage and, in industrial situations, the wastes may need to be pre-treated so that the biological elements of the wetlands can function effectively with the effluent. CW's are practical alternatives to conventional treatment of domestic sewage, industrial and agricultural wastes, storm water runoff, and acid mining drainage.

2. Types of Constructed Wetlands

Constructed wetlands for wastewater treatment can be categorized as either Free Water Surface (FWS) or Subsurface Flow (SSF) systems.

i. Free Water Surface Systems (FWS)

These systems consist of basins or channels, with some sort of subsurface barrier to prevent seepage, soil or another suitable medium to support the emergent vegetation, and water at a relatively shallow depth flowing through the unit. The shallow water depth, low flow velocity, and presence of the plant stalks and litter regulate water flow and, especially in long, narrow channels minimize short circuiting. In FWS systems, the flow of water is above the ground, and plants are rooted in the sediment layer at the base of water column (Figure 1).

ii. Subsurface Flow Systems (SSF)

These systems are essentially horizontal trickling filters when they use rock media. They have the added component of emergent plants with extensive root systems within the media. Systems using sand or soil media are also used. In SSF systems, water flows though a porous media such as gravels or aggregates, in which the plants are rooted (Figure 2).

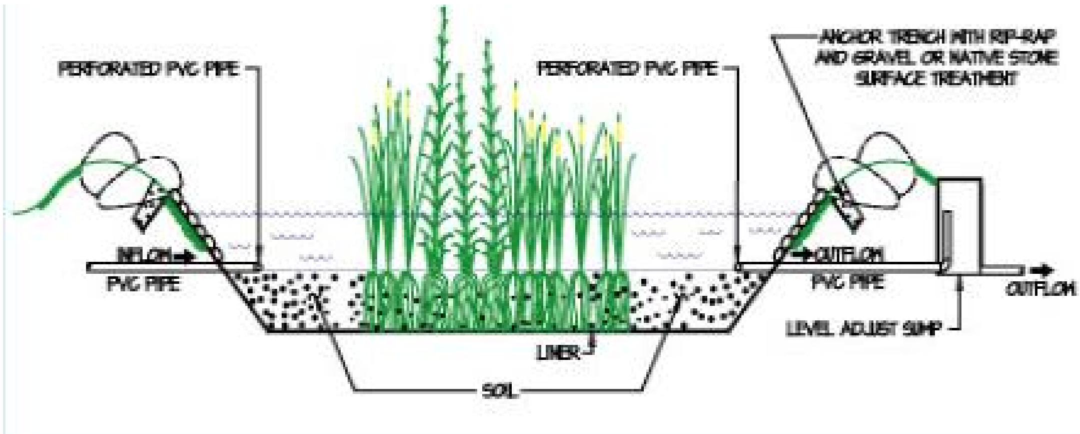


Figure 4-25: Emergent vegetation in FWS Constructed Wetlands

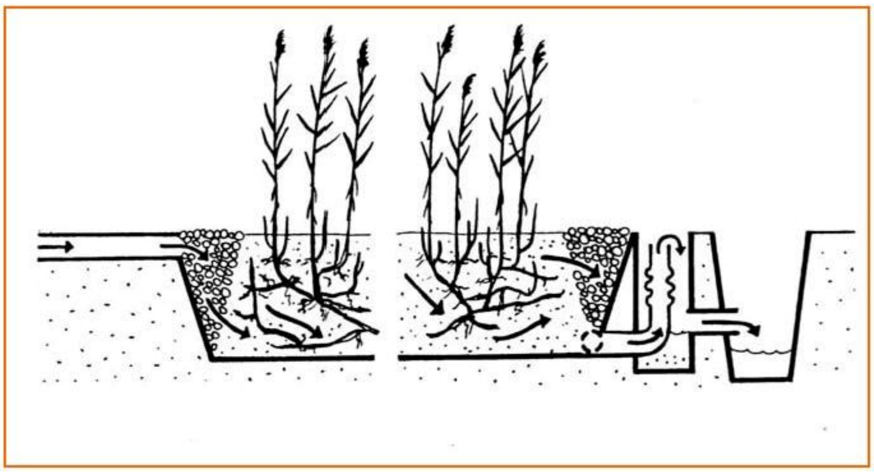


Figure 4-26: Emergent macrophyte treatment system with horizontal SSF

There are two types of SSF systems: horizontal flow SSF (HSSF) and vertical flow SSF (VSSF).

FWS systems are very appropriate for polishing secondary and tertiary effluents, and for providing habitat. The environment in the FWS systems is generally aerobic at, and near, the surface, tending toward anoxic conditions near the bottom sediment. The microbial film grows on all available plant surfaces, and is the main mechanism of pollutant removal. FWS usually exhibits more biodiversity than does SSF systems.

The objective of using CWs is to remove organic matter, suspended solids, pathogenic organisms, and nutrients such as ammonia and other forms of nitrogen and phosphorus. The growing interest in wetland system is due in part to recognition that natural systems offer advantages over conventional activated sludge and trickling filter systems.

3. Advantages and Disadvantages of Constructed Wetlands

Constructed wetlands are (1) relatively inexpensive to construct and operate, (2) easy to maintain, (3) provide effective and reliable wastewater treatment, (4) relatively tolerant of fluctuating hydrologic and contaminant loading rates (optimal size for anticipated waste load), and (5) provide indirect benefits such as green space, wildlife habitats and recreational and educational areas. The disadvantages are (1) the land requirements (cost and availability of suitable land), (2) current imprecise design and operation criteria, (3) biological and hydrological complexity and our lack of understanding of important process dynamics, (4) the costs of gravel or other fills, and site grading during the construction period, and (5) possible problems with pests.

4. Site Selection

i. Topography

A constructed wetland can be constructed almost anywhere. The emergent plant species used can tolerate winter freezing much better than aquatic plant systems. In Ontario, experimental systems have been built in heavy clay soils (Listowel) and in an abandoned mine-tailing basin (Cobalt). Because grading and excavating represent a major cost factor, topography is an important consideration in the selection of an appropriate site.

ii. Soil Permeability for Free Water Surface Systems

In selecting a site for free water surface wetland the underlying soil permeability must be considered. The most desirable soil permeability is 10^{-6} to 10^{-7} m/s (0.14-0.014 in/hr). Sandy clays and silty clay loams can be suitable when compacted. Sandy soils are too permeable to support wetland vegetation unless there is a restrictive layer in the soil profile that would result in a perched high ground water table. Highly permeable soils can be used for small wastewater flows by forming narrow trenches and lining the trench walls and bottom with clay or an artificial liner. In heavy clay soils, additions of peat moss or top soil will improve soil permeability and accelerate initial plant growth.

iii. Hydrological Factors

The performance of any constructed wetland system is dependent upon the system hydrology as well as other factors. Precipitation, infiltration, evapotranspiration (ET), hydraulic loading rate, and water depth can all affect the removal of organics, nutrients, and trace elements not only by altering the detention time, but also by either concentrating or diluting the wastewater. A hydrologic budget should be prepared to properly design a constructed wetland treatment system. Changes in the detention time or water volume can significantly affect the treatment performance (4).

For a constructed wetland, the water balance can be expressed as follows:

$$Q_i - Q_o + P - ET = [\frac{dV}{dt}] \qquad 4.67$$

Where, Q_i - influent wastewater flow,

Q_o - effluent wastewater flow,

P - precipitation, volume/time

ET - evapotranspiration, volume/time

V - volume of water, and

t – time

Ground-water inflow and infiltration are excluded from the above equation because of the impermeable barrier. Historical climatic records can be used to estimate precipitation and evapotranspiration. Empirical methods such as the Thornthwaite equation can be used to estimate evapotranspiration. Pan evaporation measurements may be useful if the wetlands will contain a significant percentage of open water areas. Then, if the system operates at a relatively constant water depth (dV/dt = 0), the effluent flow rate can be estimated using the above equation.

5. Design Approaches for Constructed Wetlands

The characteristic wastewater parameters to be treated by the CW include BOD, COD, suspended solids (SS), nitrogen compounds, phosphorus compounds, heavy metals, and pathogenic 40 organisms. Three approaches have been used to design constructed wetlands. An empirical approach is based on two different “rule of thumb” approaches (Reed et al., 1995; Kadlec and Knight 1996). Both Reed et al. (1995) and Kadlec & Knight (1996) consider wetlands as attached-growth biological reactors, therefore using a first-order plug flow kinetics model as the basis for their performance equations. The removal of soluble BOD in the SSFCW is due to microbial growth attached to the plant roots, stems, leaf litter and substrates. Both Reed et al. (1995) and Kadlec and Knight (1996) admit that BOD_5 removal in SSFCW can be described with first-order plug flow kinetics. First-order kinetics simply means that the rate of removal of a particular pollutant is direct proportional to the remaining concentration at any point within the wetland cell.

Two idealized mixing theories may be applied:

- Completely mixed reactor -- The concentration is the same as the effluent concentration at any point in the reactor;
- Plug flow -- The concentration of the reactant decreases along the length of the flow path through the reactor.

Plug flow obviously provides a more appropriate description of the flow pattern in constructed wetlands.

Reed's method for the design of constructed wetlands

The equations of Reed et al. (1995) are based on the first-order plug flow assumption for those pollutants that are removed primarily by biological processes, including biochemical oxygen demand (BOD), ammonia (NH_4) and nitrate (NO_3). Reed suggests separate equations for total suspended solids (TSS) and total phosphorus (TP). For the removal of pathogenic organisms in constructed wetlands, he suggests the same approach as that used for waste stabilization ponds. The design equations based on Reed et al. (1995) are as presented below:

For removal of BOD, NH_4 and NO_3 in constructed wetlands:

$$\ln\frac{C_i}{C_e} = K_T t \qquad 4.68$$

And

$$t = \frac{V_f}{Q} = \frac{LWy}{Q} = \frac{nLWy}{Q} = \frac{A_s yn}{Q} \qquad 4.69$$

The actual retention time in constructed wetlands is a function of the porosity of the substrate used, as defined by the third and fourth expression in Equation 3.7. The substrate porosity may be obtained as $n = V_v / V$. The value n is defined as the remaining cross-sectional area available for flow:

$$K_T = K_R \theta_R^{(T_W - T_R)} \qquad 4.70$$

$$A_s = LW = \frac{Qt}{yn} = \frac{Q\ln\frac{C_i}{C_e}}{K_T yn} = \frac{Q(\ln C_o - \ln C_e)}{K_T yn} \qquad 4.71$$

Alternatively

$$C_e = C_i \exp\left(\frac{-A_s K_T yn}{Q}\right) \qquad 4.72$$

And

$$HLR = \frac{100Q}{A_s} \qquad 4.73$$

Where A_s = treatment area of the wetland (m^2),

C_e = outlet effluent pollutant concentration (mg/l),

C_i = influent pollutant concentration (mg/l),
HLR = hydraulic loading rate (cm/day),
K_R = rate constant at reference temperature (day^{-1}),
K_T = Rate constant at temperature TW (day^{-1}),
L = length of the wetland (m),
n = porosity (percent, expressed as decimal fraction),
Q = average flow rate through the wetland (m^3/day),
t = hydraulic residence time (day^{-1}),
T_W = water temperature (°C),
T_R = reference temperature (°C),
V_f = volume of wetland available for water flow (m^3),
W = width of the wetland (m),
y = depth of the wetland (m),
θ_R = temperature coefficient for rate constant, and
V_v and V are the volume of the voids and total volume, respectively

The parameters for the design of the two types of constructed wetlands based on the Reed et al. (1995) equation (Table 4.7).

Table 4.7 Temperature coefficient for rate constant for Reed et al. (1995a) design equations

Parameter	BOD removal	Nitrification [b] (NH_4 – removal)	Denitrification [b] (NO_3 – removal)	Pathogen (removal)
T_R (°C)	20	20	20	20
Residual (mg/l)	6	0.2	0.2	-
For free water surface wetlands:				
K_R (day^{-1})	0.678	0.2187	1.00	2.6
θ_R	1.06	1.048	1.15	1.15
For sub-surface flow wetlands:				
K_R (day^{-1})	1.104	K_{NH} [c]	1.00	2.6
θ_R	1.06	1.048	1.15	1.19

[a] All rate coefficients are for temperature greater than 1°C;

[b] nitrification/denitrification are not possible at temperatures below 0°C;

[c] $$K_{NH} = 0.01854 + 0.3922\,(rz)^{2.6077} \qquad 4.74$$

Where K_{NH} = SSF nitrification rate constant (day^{-1}), and

rz = depth of bed occupied by root zone (percent, expressed as decimal fraction)

For TSS removal:

In SSF wetlands

$$C_e = C_i(0.1058 + 0.0011 HLR) \qquad 4.75$$

In FWS wetlands

$$C_e = C_i(0.1139 + 0.00213 HLR) \qquad 4.76$$

For Pathogen Removal:

Reed argues that the mechanisms for pathogen removal are essentially the same in both waste stabilization ponds and constructed wetlands. Where pathogen removal has been investigated in constructed wetlands, Equation 4.72 has proven to be conservative; hence, it is useful as a predictor:

$$C_e = \frac{C_i}{(1 + tK_T)^n} \qquad 4.77$$

Where C_e = effluent faecal coliform concentration (number/100 mL),

C_o = influent faecal coliform concentration (number/100 mL),

KT = temperature dependent rate constant (day^{-1}),

n = number of cells in series, and

t = hydraulic residence time (days)

The reliance of this formula on the number of cells in series tends to suggest that, for optimal pathogen removal, the number of cells should be maximised.

For TP removal:

In both SSF and FWS wetlands:

$$C_e = C_i * e^{\left(\frac{K_p}{HLR}\right)} \qquad 4.78$$

Where K_p is the first order phosphorous reaction rate (= 2.73cm/day)

Note that the cross-sectional area of the flow is then calculated as:

$$A_c = \frac{Q}{k_s S} \qquad 4.79$$

Where A_c - the cross-sectional area of wetland bed (d*W) normal to the direction of flow (m^2),

d - the depth (m),

ks - the hydraulic conductivity of the medium (m^3/m^2day), and

S - the slope of the bed or hydraulic gradient (as a fraction or decimal)

The bed width is then calculated as follows:

$$W = \frac{A_c}{d} \qquad 4.80$$

The cross sectional area and bed width are determined on the basis of Darcy's law, as follows:

$$Q = k_s A_s S \qquad 4.81$$

The bed cross-sectional area and bed width is independent of temperature and organic loading, since they are controlled by the hydraulic characteristics of the media.

Table 4.8: Overview of pollutant removal mechanisms in FWS wetland

Pollutant	Removal Processes
Organic material (measured as BOD)	Biological degradation, sedimentation, microbial uptake
Organic contaminants (e.g., pesticides)	Adsorption, volatilization, photolysis, and biotic/abiotic degradation
Suspended solids	Sedimentation, filtration
Nitrogen	Sedimentation, nitrification/Denitrification, microbial uptake, volatilization
Phosphorous	Sedimentation, filtration, adsorption, plant and microbial uptake
Pathogens	Natural die-off, sedimentation, filtration, predation, UV degradation, adsorption
Heavy metals	Sedimentation, adsorption, plant uptake

Water Depth in FWS Systems

The water level in the system and the duration of flooding can be important factors for the selection and maintenance of wetland vegetation. Water depth can range 4 to 18in . For warm water conditions the range is from 4 to 8in.

6. Pre-Application Treatment

To reduce capital and operating costs, minimal pretreatment of wastewater prior to discharge to a wetland is desirable. However, the level of pretreatment will also influence the quality of the final marsh effluent, and therefore effluent quality objectives must be considered. Preceding wetland treatment with a conventional primary treatment plant is capital intensive and impractical unless such a facility is already in existence. Based on studies at Listowel, some reduction of SS and BOD is desirable to reduce oxygen demand and prevent sludge accumulations in the upper reaches of the marsh. Phosphorus reduction by chemical addition is recommended in the pretreatment step when phosphorus is required.

7. Vegetation

The major benefit of plants is the transferring of oxygen to the root zone. Their physical presence in the system (the stalks, roots, and rhizomes) penetrate the soil or support medium, and transport oxygen deeper than it would naturally travel by diffusion alone(1). Perhaps most important in the FWS wetlands are the submerged portions of the leaves, stalks, and litter, which serve as the substrate for attached microbial growth. It is the responses of this attached biota that is believed responsible for much of the treatment that occurs. The emergent plants most frequently found in wastewater wetlands include cattails, reeds, rushes, bulrushes and sedges.

8. Physical Design Factors

i. System Configurations

Studies at Listowel have demonstrated the importance of a long Aspect ratio (length-to-width) to insure plug flow hydraulics. In the plug-flow, hydraulics is assumed as the major form of transport. Internal flow distribution must therefore be achieved by using high length-to-width ratios or by internal berming or barriers.

ii. Outlet Structures

The configuration of the outlet structure for a constructed wetland depends on the character of the receiving water and the number of subunits in the constructed wetland. The outlet structure for the surface flow type of wetland is shown in Figure 3-1, and includes a trench and outlet pipe with adjustable level for water level control in the wetland. Outlet structure controls must be able to control depth of water in the wetlands especially for winter ice conditions where deeper wetland conditions are required to maintain treatment levels. Outlet structures must be constructed to prevent ice damage and closed control points during freezing weather.

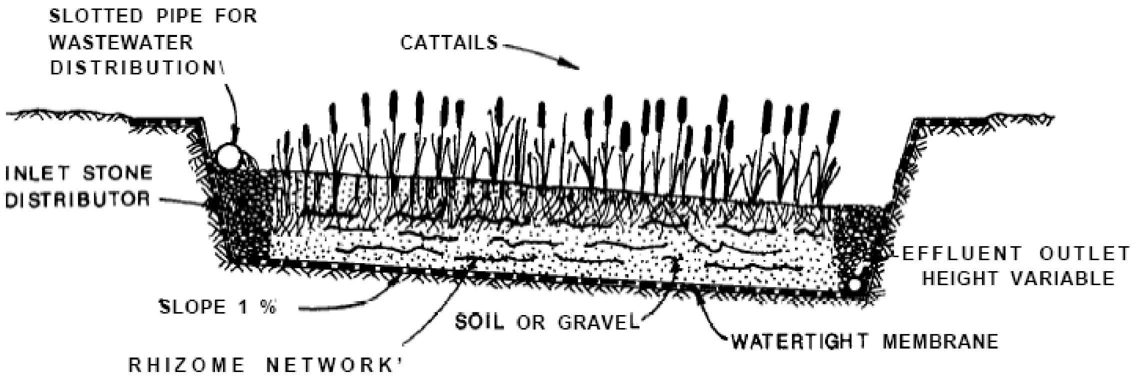


Figure 4-27: Typical cross section FWS system

iii. Vector Control in FWS Wetlands

FWS wetlands provide an ideal breeding environment for many insect pest species, particularly mosquitoes. With FWS wetlands mosquito control is essential

The method used to control mosquito includes shocking with mosquitofish, use of biological control and encouragement of predators.

iv. Harvesting of Vegetation

For free water surface systems, dry grasses are sometimes burned off annually to help maintain the hydraulic profile of the wetland, and avoid build-up of grassy hillocks, which encourage channelization. Harvesting of plant biomass is normally not regarded as a practical method for nutrient removal. Harvesting may be desirable to reduce the excessive accumulation of litter that could shorten the life span of a FWS wetland. The harvested vegetation can be used for the preparation of compost.

9. Design Procedures

The procedure for process design of FWS constructed wetlands involves the following steps:

1. Determine the limiting effluent requirements for BOD, TSS, and nitrogen or phosphorus.
2. Determine the allowable effluent BOD by subtracting 5mg/l for BOD relate to plant decay.
3. Select an appropriate apparent BOD removal rate constant and correct for the critical temperature.
4. Calculate the detention time achieve the desired level of BOD removal.
5. If BOD and TSS are the only parameters to be removed, the organic loading rate should be checked, and the large of the two areas should be selected.
6. Determine the detention time required for nitrogen or ammonia removal.
7. Select the largest detention time for design, based on the limiting design parameter.
8. Determine the required area. Increase the area by 15 to 25 % for a factor of safety.
9. Select an aspect ratio consistent with the site constraints and determine the dimensions of the wetland.
10. Check the headloss to ensure adequate head between the influent and effluent ends

4.5 Tertiary Treatment Processes

The purpose of tertiary treatment is to provide a final treatment stage to raise the effluent quality before it is discharged to the receiving environment (sea, river, lake, ground, etc.). More than one tertiary treatment process may be used at any treatment plant. If disinfection is practiced, it is always the final process. It is also called effluent polishing.

i. Filtration

Sand filtration removes much of the residual suspended matter Filtration over activated carbon, also called *carbon adsorption,* removes residual toxins

ii. Lagooning

Lagooning provides settlement and further biological improvement through storage in large man-made ponds or lagoons. These lagoons are highly aerobic and colonization by native macrophytes, especially reeds, is often encouraged. Small filter feeding invertebrates such as *Daphnia* and species of *Rotifera* greatly assist in treatment by removing fine particulates.

iii. Nutrient removal

Wastewater may contain high levels of the nutrients nitrogen and phosphorus. Excessive release to the environment can lead to a buildup of nutrients, called eutrophication, which can in turn encourage the overgrowth of weeds, algae, and cyanobacteria (blue-green algae). This may cause an algal bloom, a rapid growth in the population of algae. The algae numbers are unsustainable and eventually most of them die. The decomposition of the algae by bacteria uses up so much of oxygen in the water that most or all of the animals die, which creates more organic matter for the bacteria to decompose. In addition to causing deoxygenation, some algal species produce toxins that contaminate drinking water supplies. Different treatment processes are required to remove nitrogen and phosphorus.

iv. Nitrogen removal

The removal of nitrogen is effected through the biological oxidation of nitrogen from ammonia to nitrate (nitrification), followed by denitrification, the reduction of nitrate to nitrogen gas. Nitrogen gas is released to the atmosphere and thus removed from the water.

Nitrification itself is a two-step aerobic process, each step facilitated by a different type of bacteria. The oxidation of ammonia (NH_3) to nitrite (NO_2^-) is most often facilitated by *Nitrosomonas* (nitroso referring to the formation of a nitroso functional group). Nitrite oxidation to nitrate (NO_3^-), though traditionally believed to be facilitated by *Nitrobacter* (nitro referring

the formation of a nitro functional group), is now known to be facilitated in the environment almost exclusively by *Nitrospira* spp.
Denitrification requires anoxic conditions to encourage the appropriate biological communities to form. It is facilitated by a wide diversity of bacteria. Sand filters, lagooning and reed beds can all be used to reduce nitrogen, but the activated sludge process (if designed well) can do the job the most easily. Since denitrification is the reduction of nitrate to dinitrogen gas, an electron donor is needed. This can be, depending on the wastewater, organic matter (from faeces), sulfide, or an added donor like methanol. The sludge in the anoxic tanks (denitrification tanks) must be mixed well (mixture of recirculated mixed liquor, return activated sludge [RAS], and raw influent) e.g. by using submersible mixers in order to achieve the desired denitrification.
Sometimes the conversion of toxic ammonia to nitrate alone is referred to as tertiary treatment.
Many sewage treatment plants use axial flow pumps to transfer the nitrified mixed liquor from the aeration zone to the anoxic zone for denitrification. These pumps are often referred to as *Internal Mixed Liquor Recycle* (IMLR) pumps.

v. Phosphorus removal

Phosphorus removal is important as it is a limiting nutrient for algae growth in many fresh water systems. (For a description of the negative effects of algae, *see* Nutrient removal). It is also particularly important for water reuse systems where high phosphorus concentrations may lead to fouling of downstream equipment such as reverse osmosis.
Phosphorus can be removed biologically in a process called enhanced biological phosphorus removal. In this process, specific bacteria, called polyphosphate accumulating organisms (PAOs), are selectively enriched and accumulate large quantities of phosphorus within their cells (up to 20 percent of their mass). When the biomass enriched in these bacteria is separated from the treated water, these biosolids have a high fertilizer value.
Phosphorus removal can also be achieved by chemical precipitation, usually with salts of iron (e.g. ferric chloride), aluminum (e.g. alum), or lime. This may lead to excessive sludge production as hydroxides precipitates and the added chemicals can be expensive. Chemical phosphorus removal requires significantly smaller equipment footprint than biological removal, is easier to operate and is often more reliable than biological phosphorus removal. Another method for phosphorus removal is to use granular laterite.
Once removed, phosphorus, in the form of a phosphate-rich sludge, may be stored in a land fill or resold for use in fertilizer.

vi. Disinfection

The purpose of disinfection in the treatment of waste water is to substantially reduce the number of microorganisms in the water to be discharged back into the environment. The effectiveness of disinfection depends on the quality of the water being treated (e.g., cloudiness, pH, etc.), the type of disinfection being used, the disinfectant dosage (concentration and time), and other environmental variables. Cloudy water will be treated less successfully, since solid matter can shield organisms, especially from ultraviolet light or if contact times are low. Generally, short contact times, low doses and high flows all militate against effective disinfection. Common methods of disinfection include ozone, chlorine, ultraviolet light, or sodium hypochlorite.Chloramine, which is used for drinking water, is not used in waste water treatment because of its persistence.

Chlorination remains the most common form of waste water disinfection in North America due to its low cost and long-term history of effectiveness. One disadvantage is that chlorination of residual organic material can generate chlorinated-organic compounds that may be carcinogenic or harmful to the environment. Residual chlorine or chloramines may also be capable of chlorinating organic material in the natural aquatic environment. Further, because residual chlorine is toxic to aquatic species, the treated effluent must also be chemically dechlorinated, adding to the complexity and cost of treatment.

Ultraviolet (UV) light can be used instead of chlorine, iodine, or other chemicals. Because no chemicals are used, the treated water has no adverse effect on organisms that later consume it, as may be the case with other methods. UV radiation causes damage to the genetic structure of bacteria, viruses, and other pathogens, making them incapable of reproduction. The key disadvantages of UV disinfection are the need for frequent lamp maintenance and replacement and the need for a highly treated effluent to ensure that the target microorganisms are not shielded from the UV radiation (i.e., any solids present in the treated effluent may protect microorganisms from the UV light). In the United Kingdom, UV light is becoming the most common means of disinfection because of the concerns about the impacts of chlorine in chlorinating residual organics in the wastewater and in chlorinating organics in the receiving water. Some sewage treatment systems in Canada and the US also use UV light for their effluent water disinfection.

Ozone (O_3) is generated by passing oxygen (O_2) through a high voltage potential resulting in a third oxygen atom becoming attached and forming O_3. Ozone is very unstable and reactive and oxidizes most organic material it comes in contact with, thereby destroying many pathogenic microorganisms. Ozone is considered to be safer than chlorine because, unlike chlorine which has to be stored on site (highly poisonous in the event of an accidental release), ozone is generated onsite as needed. Ozonation also produces fewer disinfection by-products than chlorination. A disadvantage of ozone disinfection is the high cost of the ozone generation equipment and the requirements for special operators.

vii. Odour Control

Odours emitted by sewage treatment are typically an indication of an anaerobic or "septic" condition.Early stages of processing will tend to produce smelly gases, with hydrogen sulfide being most common in generating complaints. Large process plants in urban areas will often treat the odours with carbon reactors, a contact media with bio-slimes, small doses of chlorine, or circulating fluids to biologically capture and metabolize the obnoxious gases.Other methods of odour control exist, including addition of iron salts, hydrogen peroxide, calcium nitrate, etc. to manage hydrogen sulfide levels.

5- SEWAGE EFFLUENT DISPOSAL TECHNIQUES

Before we discuss, in our next chapter, the various treatments that may be given to raw sewage before disposing it off, it shall be worthwhile to first discuss the various methods and sources of disposal of sewage. The study of the sources of disposal is important, because the amount of treatment required to be given to sewage depends very much upon the source of disposal, its quality and capacity to tolerate the impurities present in the sewage effluents, without itself is getting potentially polluted or becoming less useful.

There are two general methods of disposing of the sewage effluents:

(a) Dilution i.e. disposal in water; and

(b) Effluent Irrigation or Broad Irrigation or Sewage Farming, i.e. disposal on land

5.1 Disposal by Dilution and Oxygen Sag Curve

Disposal by dilution is the process whereby the treated sewage or the effluent from the sewage treatment plant is discharged into a river stream, or a large body of water, such as a lake or sea. The discharged sewage, in due course of time, is purified by what is known as self purification process of natural waters. The degree and amount of treatment given to raw sewage before disposing it off into the river-stream in question, will definitely depend not only upon the quality of raw sewage but also upon the self purification capacity of the river stream and the intended use of its water.

Conditions Favoring Disposal by Dilution

The dilution method for disposing of the sewage can favorably be adopted under the following conditions:

(i) When sewage is comparatively fresh (4 to 5 hr old), and free from floating and settleable solids, (or are easily removed by primary treatment).

(ii) When the diluting water (i.e. the source of disposal) has high dissolved oxygen (DO) content.

(iii) Where diluting waters are not used for the purpose of navigation or water supply for at least some reasonable distance on the downstream from the point of sewage disposal.

(iv) Where the flow currents of the diluting waters are favorable, causing no deposition, nuisance or destruction of aquatic life. It means that swift forward currents are helpful, as they easily carry away the sewage to the points of unlimited dilution. On the other hand, slow back currents tend to cause sedimentation, resulting in large sludge deposits.

(v) When the outfall sewer of the city or the treatment plant is situated near some natural waters having large volumes.

5.1.1 Dilution in Rivers and Self Purification of Natural Streams

When sewage is discharged into a natural body of water, the receiving water gets polluted due to waste products, present in sewage effluents. But the conditions do not remain so for ever, because the natural forces of purification, such as dilution, sedimentation, oxidation-reduction in sun-light, etc., go on acting upon the pollution elements, and bring back the water into its original condition. This automatic purification of polluted water, in due course, is called the self-purification phenomenon. However, if the self-purification is not achieved successfully either due to too much of pollution discharged into it or due to other causes, the river water itself will get polluted, which, in turn, may also pollute the sea where the river outfalls.

The various natural forces of purification which help in affecting self-purification process are summarized below:

1. Physical forces are:
   (i) Dilution and dispersion,
   (ii) Sedimentation,
   (iii) Sunlight (acts through bio-chemical reactions).
2. Chemical forces aided by biological forces (called bio chemical forces) are:
   (iv) Oxidation (Bio),
   (v) Reduction.

These forces are described below:

(i) Dilution and Dispersion

When the putrescible organic matter is discharged into a large volume of water contained in the river-stream, it gets rapidly dispersed and diluted. The action, thus, results in diminishing the concentration of organic matter, and thus reduces the potential nuisance of sewage.

When sewage of concentration C_s flows at a rate Q_s in to a river stream with concentration C_R flowing at a rate Q_R, the concentration C of the resulting mixture is given by

$$C_sQ_s + C_RQ_R = C(Q_s + Q_R) \qquad 5.1$$

$$C = \frac{C_sQ_s + C_RQ_R}{Q_s + Q_R} \qquad 5.2$$

This equation is applicable separately to concentrations of different impurities, such as, oxygen content, BOD, suspended sediments, and other characteristic contents of sewage.

(ii) Sedimentation

The settleable solids, if present in sewage effluents, will settle down into the bed of the river, near the outfall of sewage, thus, helping in the self purification process.

(iii) Sun-light

The sun light has a bleaching and stabilizing effect of bacteria. It also helps certain micro-organisms to derive energy from it, and convert themselves into food for other forms of life, thus absorbing carbon dioxide and releasing oxygen by a process known as photo synthesis. The evolution of oxygen in river water due to sunlight will help in achieving self-purification through oxidation.

(iv) Oxidation

The oxidation of the organic matter present in sewage effluents will start as soon as the sewage outfalls into the river water containing dissolved oxygen. The deficiency of oxygen so created, will be filled up by the atmospheric oxygen. The process of oxidation will continue till the organic matter has been completely oxidized. This is the most important action responsible for effecting self purification of rivers.

(v) Reduction

Reduction occurs due to hydrolysis of organic matter settled at the bottom either chemically or biologically. Anaerobic bacteria will help in splitting the complex organic constituents of sewage into liquids and gases, and thus paving the way for their ultimate stabilization by oxidation.

The various factors on which these natural forces of purification depend are:

(a) Temperature

(b) Turbulence

(c) Hydrograph such as the velocity and surface expanse of the river-stream

(d) Available dissolved oxygen, and the amount and type of organic matter present

(e) Rate of re-aeration, etc.

Besides affecting the dilution and sedimentation rates, the temperature also affects the rate of biological and chemical activities, which are enhanced at higher temperatures and depressed at lower temperatures. The dissolved oxygen content of water, which is very essential for maintaining aquatic life and aerobic conditions (so as to avoid the anaerobic decomposition and subsequent nuisance), is also influenced by temperature. At higher temperatures, the capacity to maintain the DO concentration is low; while the rate of biological and chemical activities are high, causing thereby rapid depletion of DO This is likely to lead to anaerobic conditions, when the pollution due to putrescible organic matter is heavy.

The turbulence in the body of water helps in breaking the surface of the stream or lake, and helps in rapid re-aeration from the atmosphere. Thus, it helps in maintaining aerobic conditions in the river stream, and in keeping it clear. Too much of turbulence, however, is not desirable, because it scours the bottom sediment, increases the turbidity, and retards algae growth, which is useful in re-aeration process. Wind and undercurrents in lakes and oceans cause turbulences which affect their self-purification.

The hydrograph affects the velocity and surface expanse of the river stream. High velocities cause turbulence and rapid re-aeration, while large surface expanse (for the same cubic contents) will also have the same effects.

The larger the amount of dissolved oxygen presents in water, the better and earlier the self-purification will occur.

The amount and the type of organic matter and biological growth present in water will also affect the rate of self-purification. Algae which absorbs carbon dioxide and gives out oxygen, is thus, very helpful in the self-purification process.

The rate of re-aeration i.e. the rate at which the DO deficiency is replenished, will considerably govern the self-purification process. The greater is this rate, the quicker will be the self-purification, and there will be no chances of development of anaerobic conditions.

1. Zones of Pollution in a River-Stream

A polluted stream undergoing self-purification can be divided into the following four zones:

- (i) Zone of degradation
- (ii) Zone of active decomposition
- (iii) Zone of recovery; and
- (iv) Zone of cleaner water

These zones are discussed below:

(i) Zone of degradation or Zone of pollution

This zone is found for a certain length just below the point where sewage is discharged into the river-stream. This zone is characterized by water becoming dark and turbid with formation of sludge deposits at the bottom. DO is reduced to about 40% of the saturation value (Saturation value at 30°c = 7.6 mg/l). There is an increase in carbon dioxide content; re-oxygenation (i.e. re-aeration) occurs but is slower than de-oxygenation.

These conditions are unfavorable to the development of aquatic life; and as such, algae die out, but fish life may be present feeding on fresh organic matter. Moreover, certain typical bottom worms such as Limondrilus and Tubifex appear with sewage fungi, such as sphaerotilusnatans.

(ii) Zone of active decomposition

This zone is marked by heavy pollution. It is characterized by water becoming greyish and darker than the previous zone. DO concentration falls down to zero and anaerobic conditions may set in with the evolution of gases like methane, carbon dioxide, hydrogen sulphide, etc., bubbling to the surface, with masses of sludge forming an ugly scum layer at the surface. As the organic decomposition slackens due to stabilization of organic matter, the re-aeration sets in and DO again rises to the original level (i.e. about 40%).

In this zone, bacteria flora will flourish. At the upper end, anaerobic bacteria will replace aerobic bacteria, while at its lower end, the position will be reversed. Protozoa and fungi will first disappear and then reappear. Fish life will be absent. Algae and Tubifex will also mostly be absent. Maggots and Psychoda (sewage fly) larvae will, however, be present in all but the most septic sewage.

(iii) Zone of recovery

In this zone, the river stream tries to recover from its degraded condition to its former appearance. The water becomes clearer, and so the algae reappear while fungi decrease. BOD falls down and DO content rises above 40% of the saturation value; Protozoa, Rotifers, Crustaceans and large plants like Sponges, Bryozons, etc. also reappear. Bottom organisms will include: Tubifex, Mussels, Snails, etc. The organic material will be mineralized to form nitrates, sulphates, phosphates, carbonates, etc.

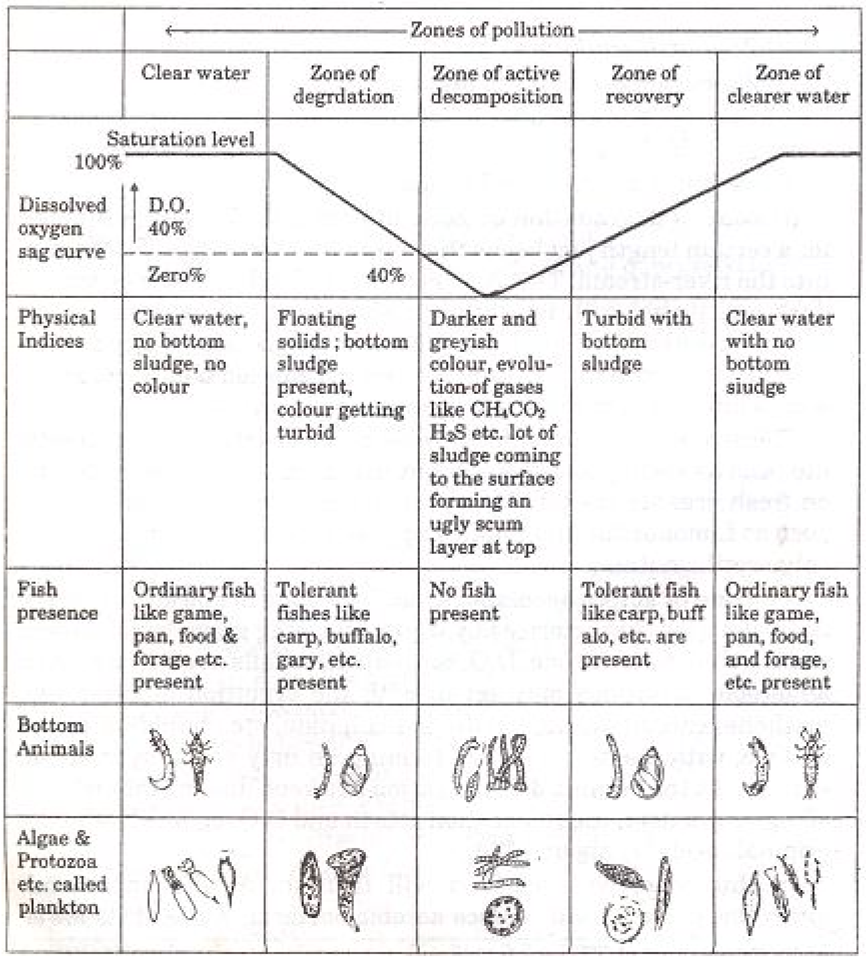
(iv) Zone of cleaner water

In this zone, the river attains its original conditions with DO rising up to the saturation value. Water becomes attractive in appearance and Game fish (which requires at least 4 to 5 mg/l of DO) and usual aquatic life prevails. Same pathogenic organisms may still, however, survive and remain present, which confirms the fact that "when once river water has been polluted, it will not be safe to drink it, unless it is properly treated."

2. Indices of Self-Purification

The stage of self-purification process can be determined by the physical, chemical and biological analysis of the water. Color and turbidity are the physical indices, while DO, BOD and suspended solids are the chemical indices which can mark the stages of purification. Moreover, the biological growth present in water can also indicate the stage of purification process, as different types of micro and macro organisms will exist in polluted water under different conditions, as discussed in the previous sub article.

The different zones of pollution (i.e. various stages in the self-purification process) and the physical, chemical and biological indices, characteristics of each zone, are shown in Figure 5-1:.



	Zones of pollution				
	Clear water	Zone of degrdation	Zone of active decomposition	Zone of recovery	Zone of clearer water
Dissolved oxygen sag curve	Saturation level 100% D.O. 40% Zero%	40%			
Physical Indices	Clear water, no bottom sludge, no colour	Floating solids ; bottom sludge present, colour getting turbid	Darker and greyish colour, evolution of gases like CH_4CO_2 H_2S etc. lot of sludge coming to the surface forming an ugly scum layer at top	Turbid with bottom sludge	Clear water with no bottom siudge
Fish presence	Ordinary fish like game, pan, food & forage etc. present	Tolerant fishes like carp, buffalo, gary, etc. present	No fish present	Tolerant fish like carp, buffalo, etc. are present	Ordinary fish like game, pan, food, and forage, etc. present
Bottom Animals					
Algae & Protozoa etc. called plankton					

Figure 5-1: Showing zones of pollution along a river stream

3. The Oxygen Deficit of a Polluted River-Stream

The oxygen deficit D at any time in a polluted river-stream is the difference between the actual DO content of water at that time and the saturation DO content (The normal saturation DO value for fresh water varies between 14.6mg/l to 7.6mg/l for temperature varying between 0°c to 30°c.) at the water temperature; i.e.

$$\text{Oxygen deficit } (D) = \text{Saturation DO} - \text{Actual DO}$$

In order to maintain clean conditions in a river-stream, the oxygen deficit must be nil, and this can be found out by knowing the rates of de-oxygenation and re-oxygenation.

De-oxygenation Curve

In a polluted stream, the DO content goes on reducing due to decomposition of volatile organic matter. The rate of de-oxygenation depends upon the amount of the organic matter remaining to be oxidized at the given time (i.e. L_t) as well as on the temperature of reaction (i.e. T). Hence, at a given temperature, the curve showing depletion of DO with time, i.e. deoxygenation curve

(Refer curve I of Figure 5-2:) is similar to the first stage BOD curve (Refer Figure 2-4). It can also be expressed mathematically as per Eq. (2.3).

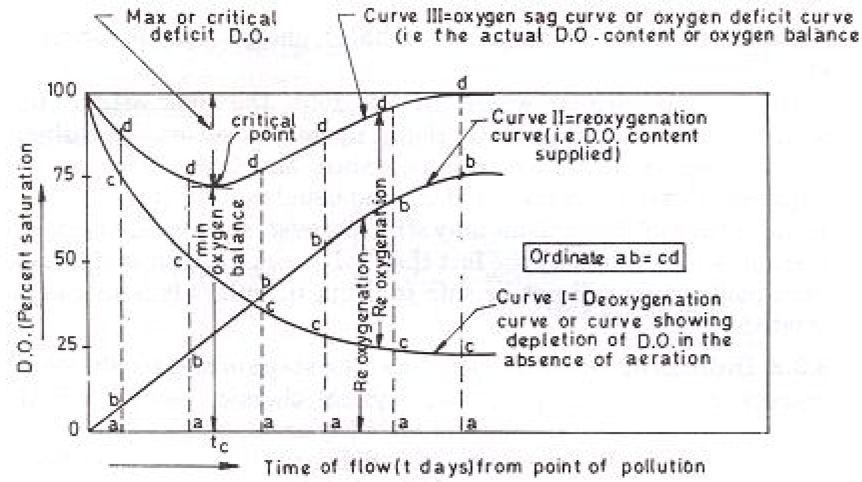


Figure 5-2: Deoxygenation curve

Re-oxygenation Curve

In order to counter-balance the consumption of DO due to de-oxygenation, atmosphere supplies oxygen to the water and the process is called re-oxygenation. The rate at which the oxygen is supplied by the atmosphere to the polluted water depends upon:

(i) the depth of the receiving water (rate is more in a shallow depth)

(ii) the condition of the body of water (rate is more in a running stream than in a quiescent pond)

(iii) the saturation deficit or the oxygen deficit (i.e. the deficit of DO below the saturation value); and

(iv) the temperature of water

Depending upon these factors, the rate of re-oxygenation can also be expressed mathematically and plotted in the form of a curve called re-oxygenation curve (Refer curve II-Figure 5-2:).

Oxygen Deficit Curve

In a running polluted stream exposed to the atmosphere, the de-oxygenation as well as the re-oxygenation go hand in hand. If de-oxygenation is more rapid than the re-oxygenation, an oxygen deficit will result.

Note: If the DO content becomes zero, aerobic conditions will no longer be maintained and putrefaction will set in.

The amount of resultant oxygen deficit can be obtained by algebraically adding the de-oxygenation and re-oxygenation curves (see curve III-Figure 5-2:). The resultant curve so obtained is called ***the oxygen sag curve*** or the ***oxygen deficit curve***. From this curve, the oxygen deficit and oxygen balance (i.e. 100 - D) percent in a stream after a certain lapse of time, can be found out.

It can also be seen that when the de-oxygenation rate exceeds the re-oxygenation rate, the oxygen sag curve shows increasing deficit of oxygen; but when both the rates become equal, the critical point is reached, and then finally when the rate of de-oxygenation falls below that of re-oxygenation, the oxygen deficit goes on decreasing till becoming zero.

The entire analysis of super-imposing the rates of de-oxygenation and re-oxygenation have been carried out mathematically, and the obtained results expressed in the form of famous Streeter-Phelps equation; i.e.,

$$D_t = \frac{K_D * L}{K_R - K_D} * [(10)^{-K_D * .t} - (10)^{-K_R * t}] + [D_0 * (10)^{-K_R * t}] \qquad 5.3$$

Where, D_t = the DO deficit in mg/l after t days.

L = Ultimate first stage BOD of the mix at the point of waste discharge

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

K_D = De-oxygenation coefficient for the wastewater, which can be considered as equal to the BOD rate constant determined in the laboratory through BOD tests performed at different times on BOD bottles. K_D varies with temperature as:

$$K_{D(T)} = K_{D(20)} * (1.047)^{T-20°} \qquad 5.4$$

The typical values of $K_{D(20)}$ vary between 0.1 to 0.2, generally taken as 0.1.

K_R = Re-oxygenation coefficient for the stream.

Where, v = Average stream velocity in m/s, y = Average stream depth in m

K_R varies with temperature as per the equation:

$$K_{R(T)} = K_{R(20)} * (1.016)^{T-20°} \qquad 5.5$$

Where, $K_{R(T)}$ is the K_R value at T°c and $K_{R(20)}$ is the K_R value at 20°c. Typical values of $K_{R(20)}$ are given in Table 5-1.

Table 5-1 Values of Re-oxygenation Coefficient (K_R) at 20°c

S.No.	Type of water body	Value of $K_{R(20)}$ per day
1	Small ponds and back waters	0.05 - 0.10
2	Sluggish streams, large lakes and impounding reservoirs	0.10 - 0.15
3	Large stream of low velocity	0.15 - 0.20
4	Large streams of normal velocity	0.20 - 0.30
5	Swift streams	0.30 - 0.50
6	Rapids and waterfalls	Over 0.5

The oxygen deficit curve can be plotted easily with the help of Eq. (5.3), by using different values of t in days:

The critical time (t_c) after which the minimum dissolved oxygen occurs can be found by differentiating Eq. (5.3) and equating it to zero; which on solving gives

$$t_c = \left[\frac{1}{K_R - K_D}\right] \log\left(\left\{\frac{K_D * L - K_R * Do + K_D * D_0}{K_D * L}\right\} * \frac{K_R}{K_D}\right) \qquad 5.6$$

and the critical or maximum oxygen deficit is given by

$$D_c = \frac{K_D * L}{K_R}(10)^{-K_D * t_c} \qquad 5.7$$

The constant K_R / K_D is sometimes represented by f, called self-purification constant, the values of which are given in Table 5-2.

Table 5-2 Values of self-purification constant ($f = K_R / K_D$)

S.No	Type of water body	Value of 'f'
1	Small ponds and back waters	0.5 - 1.0
2	Sluggish streams, large lakes and impounding reservoirs	1.0 - 1.5
3	Large stream of low velocity	1.5 - 2.0
4	Large streams of normal velocity	2.0 - 3.0
5	Swift streams	3.0 - 5.0
6	Rapids and waterfalls	Over 5.0

Using $\frac{K_R}{K_D}$ as f, the Eq. 5.6 becomes as

$$t_c = \frac{1}{K_D(f-1)} \log\left(\left\{1 - (f-1) * \frac{D_0}{L}\right\} * f\right) \qquad 5.8$$

and equation 5.7 becomes

$$D_c = \frac{L}{f}(10)^{-K_D * t_c} \qquad 5.9$$

Taking log, we get

$$\log D_c = \log\frac{L}{f} - K_D * t_c$$

Substituting the value of t_c from Eq. (8.8) in Eq. (8.10), we get

$$\log D_c = \log\frac{L}{f} - \frac{K_D}{K_D(f-1)} \log\left(\left\{1 - (f-1) * \frac{D_0}{L}\right\} * f\right)$$

$$\log D_c = \log\frac{L}{f} - \frac{1}{(f-1)} \log\left(\left\{1 - (f-1) * \frac{D_0}{L}\right\} * f\right)$$

$$(f-1)\left(\log\frac{L}{f} - \log D_c\right) = \log\left(\left\{1 - (f-1) * \frac{D_0}{L}\right\} * f\right)$$

$$\log\left(\frac{\frac{L}{f}}{D_c}\right)^{(f-1)} = \log\left(\left\{1-(f-1)*\frac{D_0}{L}\right\}*f\right)$$

$$\left(\frac{L}{D_c*f}\right)^{(f-1)} = f*\left(1-(f-1)*\frac{D_0}{L}\right) \qquad 5.10$$

This is the important first stage equation in which L is the BOD of the mixture of sewage and stream, and f (K_D and K_R also) corresponds to the temperature of the mixture of sewage and stream at the outfall.

The above equations are of practical value in predicting the oxygen content at any point along a stream, and thus help us in estimating the degree of waste treatment required, or of the amount of dilution necessary, in order to maintain a certain DO in the stream.

Example 5-1

The sewage of a town is to be discharged into a river stream. The quantity of sewage produced per day is 8 million liters, and its BOD is 250mg It. If the discharge in the river is 200l/s and its BOD is 6mg/l, find out the BOD of the diluted water.

Example 5-2

A city discharges 1500 litres per second of sewage into a stream whose minimum rate of flow is 6000 litres per second. The temperature of sewage as well as water is 20°c. The 5 day BOD at 20°c for sewage is 200mg/l and that of river water is 1mg/l. The DO content of sewage is zero, and that of the stream is 90% of the saturation DO If the minimum DO to be maintained in the stream is 4.5 mg/l, find out the degree of sewage treatment, required. Assume the de-oxygenation coefficient as 0.1 and re-oxygenation coefficient as 0.3.

Example 5-3

A city discharges 100 cumecs of sewage into a river, which is fully saturated with oxygen and flowing at the rate of 1500 cumecs during its lean days with a velocity of 0.1m/sec. The 5-days BOD of sewage at the given temperature is 280mg/l. Find when and where the critical DO deficit will occur in the downstream portion of the river, and what is its amount? Assume coefficient of purification of the stream (f) as 4.0 and coefficient of de-oxygenation (K_D as 0.1).

Example 5-4

A waste water effluent of 560 l/s with a BOD = 50mg/l, DO = 3.0 mg/l and temperature of 23°c enters a river where the flow is 28m^3/sec, and BOD = 4.0 mg/l, DO = 8.2mg/l, and temperature of 17°c. K_1 of the waste is 0.10 per day at 20°c. The velocity of water in the river downstream is 0.18m/s and depth of 1.2m.

Determine the following after mixing of waste water with the river water:

(i) Combined discharge

(ii) BOD

(iii) DO

(iv) Temperature

5.1.2 Disposal of Wastewaters in Lakes and Management of Lake Waters

1. Lake Pollutants

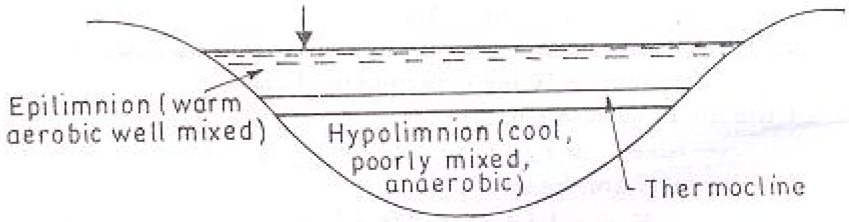
Disposal of wastewaters in confined lakes or reservoirs is much more harmful than its disposal in flowing streams and rivers. Water quality management in lakes in entirely different from that in rivers. It is infact the phosphorous (a nutrient largely contained in industrial as well as domestic wastewaters), which seriously affects the water quality of lakes; and is hence considered as the prime lake pollutant. Oxygen demanding wastes may be the other important lake pollutants. The toxic chemicals from industrial wastewaters may also sometimes very adversely affect some special classes of the lakes. However, phosphorous (a nutrient) constitutes the most important lake pollutant, and needs special study in water quality management of lakes.

A study of the lake systems is essential to understand the role of phosphorous in lake pollution. The study of lakes is called limnology.

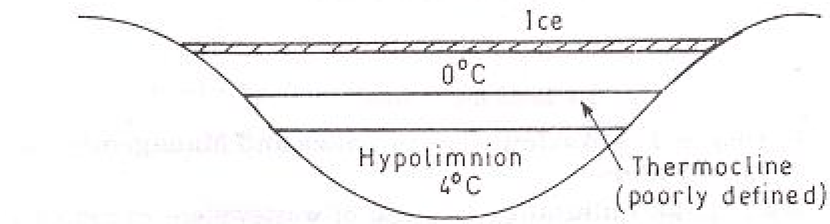
2. Stratification in Lakes

The water of a lake gets stratified during summers and winters, as discussed below:

During summer season, the surface water of a lake gets heated up by sunlight and warm air. This worm water being lighter remains in upper layers near the surface, until mixed downward by turbulence from winds, waves, boats and other forces. Since such turbulence extends only to a limited depth from below the water surface, the top layers of water in the lake become well mixed and aerobic. This warmer, well mixed and aerobic depth of water is called epilimnion zone. The lower depth, which remains cooler, poorly mixed and anaerobic, is called the hypolimnion zone. There may also exist an intermediate zone or a dividing line, called thermocline, as shown in Figure 5-3: (a).



(a) Stratification of a lake during summer.



(b) Stratification of a lake during winter.

Figure 5-3: Stratification of lakes

The change from epilimnion to hypolimnion can be experienced while swimming in a lake. When you swim in top layers horizontally, you will feel the water warmer; and if you dive deeper, you will find the water cooler. The change line will represent monocline. The depth of epilimnion zone depends upon the size of the lake for the same temperature changes. It may be as little as 1m in small lakes and may be as large as 20 m or more in large lakes. This depth also depends upon the storm activity in the spring when stratification is developing. A major storm at the right time will mix the warmer water to a substantial depth and thus create a deeper epilimnion zone than its normal depth. Once formed, lake stratification is very stable, and can only be broken by exceedingly violent storms. As a matter of fact, as summer progresses, this stability increases; the epilimnion continues to warm, while the hypolimnion remains at a fairly constant temperature.

With the onset of winter season, the epilimnion cools, until it is denser than the hypolimnion. The surface water then sinks, causing '*overturning*'. The water of the hypolimnion rises to the surface, where it cools and again sinks. The lake thus becomes completely mixed, making it quite aerobic. In regions of freezing temperatures, when the temperature drops below 4°c, the above process of overturning (or turn over) stops, because water is most dense at this temperature. Further cooling or freezing of the water surface results in winter stratification, as shown in Figure 5-3: (b).

With the passing of winters and commencement of spring season, the surface water again warms up and overturns, and lake becomes completely mixed. The lakes in regions of temperate climate will, therefore, have at least one, if not two, cycles of stratification and turn-over every year.

3. Biological Zones in Lakes

Lakes have been found to exhibit distinct zones of biological activity, largely determined by the availability of light and oxygen. The most important biological zones are:

(i) euphotic zone;
(ii) littoral zone; and
(iii) benthic zone.

These zones are shown in Fig. 8.4, and briefly discussed below:

i. Euphotic zone

The upper layer of lake water through which sunlight can penetrate, is called the euphotic zone. All plant growth occurs in this zone. In deep water, algae grow as the most important plants, while rooted plants grow in shallow water near the shore.

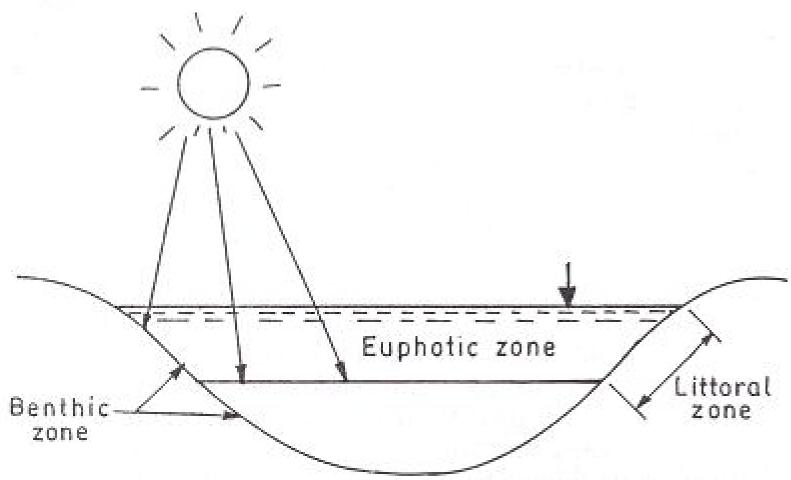


Figure 5-4: Biological zones in a lake

ii. Littoral zone

The shallow water near the shore, in which rooted plants grow, is called the littoral zone. The extent of the littoral zone depends on the slope of the lake bottom, and the depth of the euphotic zone. The littoral zone cannot extend deeper than the euphotic zone, as shown in Fig. 8.3.

iii. Benthic zone

The bottom sediments in a lake comprise what is called the benthic zone. As the organisms living in the overlying water die, they settle down to the bottom, where they are decomposed by the organisms living in the benthic zone. Bacteria are always present in this zone. The presence of higher life forms, such as worms, insects and crustaceans however, depends upon the availability of oxygen at the lake bottom.

5.1.3 Disposal of Wastewater in Sea Water

Sea water normally contains 20% less oxygen than that contained in fresh water of a river stream. Moreover, sea water normally contains a large amount of dissolved matter. As such, the capacity of sea water to absorb sewage solids is not as high as that of fresh water of a stream. Moreover, sewage solids, when thrown into sea water, chemically react with the dissolved matter of sea water, resulting in precipitating some of the sewage solids, giving a milky appearance to sea water and forming ***sludge banks***. These sludge banks are undesirable, as they are likely to produce offensive hydrogen sulphide gas by reacting with the sulphate rich water of the sea.

As pointed out earlier, the oxygen content of sea water is less than that of fresh water, and also, its re-aeration is slower. However, since the sea contains too larger volumes of water, most of these deficiencies are removed, provided the sewage is taken deep into the sea and away from the coast line.

Since the specific gravity of sea water is greater than that of sewage, and temperature of sea water is lower than that of sewage, the lighter and the warmer sewage will rise up to the surface when thrown into the sea water. This will result in spreading of the sewage at the top surface of sea in a thin film or 'sleek'.

To prevent the backing up and spreading of sewage on the sea shore, the sewage should be disposed of only during low tides. Large sized tanks may, therefore, be constructed to hold the sewage during high tides. Provision of a large sized sewer, grated with a non-return valve at the end, is also an alternative to hold the sewage during high tides.

In all, the following points should be kept in mind while discharging sewage into the sea:

(i) The sewage should be discharged in deep sea only.

(ii) In order to mix the sewage properly with sea water, the sewage should be released at a minimum depth of 3 to 5 meters below the water level, and by taking it sufficiently inside (about $1\frac{1}{2}\,km$) from the shore line. This measure will prevent the sewage from accumulating on the shore, and thus preventing nuisance to baths and recreation centres on the shores.

(iii) Before deciding the position of outfall point, the sea currents, wind direction, velocity, etc. should be thoroughly studied. The point of disposal should be such that the sewage is taken away from the shore by the-winds, and not brought back near the shore.

(iv) The outfall sewer should be placed on a firm rocky foundation, and encased in thick stone masonry, so as to properly protect it from wave action, floating debris, etc.

5.2 Land Disposal and Treatment

1. Disposal of Sewage Effluents on Land for Irrigation

In this method, the sewage effluent (treated or diluted) is generally disposed of by applying it on land. The percolating water may either join the water-table, or is collected below by a system of under drains. This method can then be used for irrigating crops.

This method, in addition to disposing of the sewage, may help in increasing crop yields (by 33% or so) as the sewage generally contains a lot of fertilizing minerals and other elements. However, the sewage effluent before being used as irrigation water must be made safe. In order to lay down the limiting standards for sewage effluents, and the degree of treatment required, it is necessary to study as to what happens when sewage is applied on to the land as irrigation water.

When raw or partly treated sewage is applied on to the land, a part of it evaporates, and the remaining portion percolates through the ground soil. While percolating through the soil, the suspended particles present in the sewage are caught in the soil voids. If proper aeration of these voids is maintained, the organic sewage solids caught in these voids get oxidized by aerobic process. Such aeration and aerobic conditions will more likely prevail, if the soil is sufficiently porous and permeable (such as sands and porous loams). However, if the land is made up of heavy, sticky and fine grained materials (such as clay, rock, etc.), the void spaces will soon get choked up, and thus resulting in non-aeration of these voids. This will lead to the developing of non-aerobic decomposition of organic matter, and evolution of foul gases. Moreover, excessive clogging may also result in ugly ponding of sewage over the farm land, where mosquitoes may breed in large number, causing further nuisance.

Application of too strong or too heavy load of sewage will also similarly result in the quick formation of anaerobic conditions. The greater is the sewage load, more likely it will be for the soil to get clogged. Hence, if the sewage load is reduced either by diluting it or by pre-treating it, it may be possible to avoid the clogging of the soil pores. The degree of treatment required will, however, considerably depend upon the type of the soil of the land. If this soil, to be irrigated, is sandy and porous, the sewage effluents may contain more solids and other wastes, and thus requiring lesser treatment, as compared to the case where the soil is less porous and sticky.

The pretreatment process may be adopted by larger cities which can afford to conduct treatment of sewage; whereas the dilution technique may be adopted by smaller cities. When sewage is diluted with water for disposal for irrigation, too large volumes of dilution water are generally not needed, so as not to require too large areas for disposal. The extent of land area required for disposing a certain volume of sewage effluent can be worked out from the values given in

Table 5-3:.

Table 5-3: Recommended Doses for Sewage Farming

Types of soil	Doses of sewage in cubic meters per hectare per day	
	Raw sewage	Settled sewage
Sandy	120 – 150	220 – 280
Sandy loam	90 – 100	170 – 220
loam	60 – 80	110 – 170
Clayey loam	40 – 50	60 – 110
clayey	30 - 45	30 - 60

Example 5-6

A town having population of 40,000 disposes sewage by land treatment. It gets a per capita assured water supply from waterworks at a rate of 130l/day. Assuming that the land used for sewage disposal can absorb 80m^3 of sewage per hectare per day, determine the land area required, and its cost at the rate of $25,000 per hectare. Make suitable assumptions where needed.

Solution

Population = 40,000

Rate of water supply = 130 l/day/person

Total water supplied per day

$$= 40{,}000 * 130\ \mathrm{l} = 5{,}200{,}000\ \text{lites} = 5{,}200\ \text{cu.m.}$$

Assuming that 80% of this water appears as sewage,

The quantity of sewage produced per day

$$= 0.8 * 5200 = 4160\ \text{cum.}$$

Therefore, area of land required for disposing sewage

$$= \frac{4160}{80} = 52\ \text{hectares}$$

Providing 50% extra land for rest and rotation,

The total land area required

$$= 1.5 * 52 = 78\ \text{hectares}$$

Cost of land involved

$$= 25{,}000 * 78 = \$1{,}950{,}000$$

Example 5-7

A town disposes sewage by land treatment. It has a sewage farm of area 150 hectares. The area included an extra provision of 50% for rest and rotation. The population of the town being

50,000 and rate of water supply 140 litres per capita per day. If 75% of the water is converted into sewage, determine the consuming capacity of the soil.

Solution

Quantity of water produced per day

$$= 50{,}000 * 140 \text{litres/day} = 7{,}000{,}000 \text{l/day} = 7{,}000 \text{ cu. m/day}$$

Quantity of sewage produced

$$= 0.75 * 7000 = 5{,}250 \text{ cu. m/day}$$

Area of farm land provided

= 150 hectares with 50% additional reserve

Hence, area provided for immediate need

$$= \frac{150}{1.5} = 100 \text{ hectares}$$

100 hectares is capable of passing 5250 cum per day

Consuming capacity of soil

$$= \frac{5250}{100} = 52.5 \text{ cu. m per hectare per day}$$

2. Quality Standards for Wastewater Effluents to be discharged on Land for Irrigation

These standards are based upon the quality of irrigation water required by the crops, and thus limit the concentrations of pollutants contained in sewage or industrial liquid wastes, which may prove harmful to the crops.

Table 5-4: Effluent standards for Irrigation under Environment (Protection) Rules, 1986

S.No.	Characteristics of effluent i.e. pollutant of waste water	Standards under Environment (Protection) Rules, 1986
(l)	(2)	(3)
1	Color and odor	All efforts should be made to remove color and unpleasant odor as far as practicable
2	BOD_5	100mg/l
3	Suspended Solids (SS)	200 mg/l
4	pH Value	5.5 to 9.0
5	Oil and grease	10 mg/l
6	Arsenic (As)	0.2 mg/l
7	Cyanide (as CN)	0.2 mg/l

The effluent irrigation method for disposal of sewage can be favorably adopted under the following conditions:

(i) When some natural rivers or water courses are not located in the vicinity, the land treatment is the only alternative left, and has to be adopted.

(ii) When irrigation water is scarcely available, the use of sewage for irrigating crops is a good alternative.

(iii) When large areas of open land are available, broad irrigation may be practised over it with the help of sewage effluents, and good returns can be earned by raising cash crops. Crops like wheat, cotton,/sugarcane, plantain, grasses, fodder, coconut, orange trees etc. have been successfully grown with advantage on sewage farms.

(iv) The method of effluent irrigation will prove useful in areas of low rainfall, as this will help in maintaining good absorption capacity of the soil.

(v) The area for land treatment or sewage farming should preferably be porous, such as sandy, loamy, or alluvial soils, or soft moorum. It should not be made of heavy retentive soils like clay, etc., which prevent easy aeration of the soil voids, and thus creating anaerobic conditions.

(vi) This method of disposal of sewage, poses problems during the periods when no irrigation water is required for the crops especially during rains. This method is, therefore preferred when sewage can be diverted to some river streams (flowing high during rainy season).

(vii) This method is preferred in areas of low water-table, where rate of percolation may be quite high.

6- SLUDGE TREATMENT AND DISPOSAL

The main objective of any type of wastewater treatment is to get clear effluent and good sludge.
The sludge generated from wastewater by primary and secondary and tertiary treatment processes must be treated and properly disposed of.
The higher the degree of wastewater treatment, the larger the quantity of sludge to be treated and handled.
A properly designed and efficiently operated sludge processing and disposal system is essential to the overall success of the wastewater treatment effort.
The main objectives of sludge treatment are, as follows:

- reduction in the volume of sludge for disposal by removing some of the water,
- stabilization of the organic matter contained in the sludge,
- destruction of pathogenic organisms,
- disposal of the sludge in a safe and aesthetically acceptable manner.

6.1 Sludge and Its Moisture Content

The sludge, which is deposited in a primary sedimentation tank, is called raw sludge; and the sludge which is deposited in a secondary clarifier is called Secondary Sludge. Raw sludge is odorous, contains highly putrescible organic matter, and is, thus, very objectionable. Secondary sludge is also putrescible, though a little less objectionable. The sludge withdrawn from the bottom of the sedimentation basins must, therefore, be stabilized before its final disposal. In addition to its putrescibility, another problem posed by the sludge is its high moisture content. In case of raw sludge, the moisture content is about 95%; in case of secondary sludge from a trickling filter plant, it is about 96 to 98%; and in case of secondary sludge from an activated sludge treatment plant, it is about 98% to over 99%. The sludge containing high moisture content becomes very, bulky, and difficult to handle. For example, sludge with 95% moisture, contains 5litres of solid matter in 100 liters of sludge. Similarly, the sludge with 90% moisture will contain 10 liters of solid matter in l00 liters of sludge, i.e. 5 liters of solid matter in 50 liters of sludge. Thus, the sludge with 90% moisture will be half in quantity as compared to that of the sludge with 95% moisture, for the same volume of solids. Similarly, sludge with 99% moisture contains 1 liter of solids in 100 liters of sludge, whereas, a sludge with 90% moisture contains 5 liters of solids in 100 liters of sludge, i.e. 1 liter of solids in 20 liters of sludge. Hence, the sludge with 99% moisture will be $\frac{100}{20} = 5$ times more bulky than the sludge with 95% moisture, for storing the same solid content.
It, therefore, follows that if the moisture content of the sludge is reduced, its volume will go on decreasing. If moisture content is reduced to about 70 to 80%, the sludge becomes viscous; and

at about 10% moisture content, it becomes dry, and assumes powder form. The complete moisture can, however, be removed only by special treatments, since the water is so tenaciously held in the sludge.

Example 6-1

A sedimentation tank is treating 4.5 million liters of sewage per day containing 275ppm of suspended solids. The tank removes 50% of suspended solids. Calculate the quantity of sludge produced per day in bulk and weight, if:

(b) moisture content of sludge is 98%

(c) moisture content of sludge is 96%

Solution

Volume of sewage treated

$= 4.5*10^6$litres/day

Since suspended solids amount to 275mg/l, the weight of suspended solids present in sewage

$$= \frac{275 * 4.5}{10^6} * 10^6 \text{ kg/day} = 1237.5\text{kg/day}$$

Since 50% of solids are removed in sedimentation tank,

The wt. of solids removed in sedimentation tank

$$= 1237.5 * \frac{50}{100} = 618.75\text{kg/day}$$

a. When moisture content of sludge is 98%, then 2kg of solids (dry sludge) will make = 100kg of wet sludge

Therefore, 618.75kg of solids (dry sludge) will make

$$= \frac{100}{2} * 618.75 = 30940\text{kg}$$

Hence, wet sludge or sludge produced per day

$$= 30{,}940 \text{ kg} = 30.94 \text{ tonnes}$$

Assuming the sp. gravity of wet sludge (sludge) as 1.02,

Unit wt. of sludge = $1.02 t/m^3$

Vol. of wet sludge produced per day

$$= \frac{\text{Wt}}{\text{unit wt}} = \frac{30.94}{1.02} = 30.33 \text{ m}^3$$

Vol. of sludge (when its moisture content is 98%) = $30.33m^3$

Hence, the vol. of sludge when its moisture content is 98% = 30.33cu.m

b. When moisture content is 96%, then 4 kg of solids will make

= 100kg of wet sludge

Therefore, 618.75kg of solids will make

$$= \frac{100}{4} * 618.75 = 15468.75\text{kg of wet sludge}$$

$\cong$ 15,470kg of wet sludge = 15.47tonnes of wet sludge

Hence, wt. of sludge (when its moisture content is 96%) = 15.47tonnes.

If sp. gravity of sludge is 1.02, then Vol. of sludge (when its moisture content is 96%)

$$= \frac{15.47}{1.02} = 15.17\text{m}^3$$

Hence, the vol. of sludge at 96% moisture content = 15.17cu. m.

Note: It shows that the sludge is reduced to half its volume when its moisture content is lowered from 98% to 96%.

Example 6-2

There is sewage sludge with volume containing a certain moisture content p_1 (percent). What will be the volume of this sludge if its moisture content is reduced to p (percent).

Solution:

Let the given sewage contains solids = W kg. Let its volume be V_1 at a moisture content of p_1 (percent), and V at a moisture content of p (percent).

At moisture content of p_1,

$(100 - p_1)$ kg of solids will make = 100kg of wet sludge

W kg of solids will make

$$= \frac{100 * W}{(100 - p_1)} \text{ kg of wet sludge}$$

Or Wt. of sludge produced

$$= \frac{100 * W}{(100 - p_1)} \text{ kg}$$

If γ_s is the unit wt. of sludge, in kg/m^3, then

Vol. of sludge produced

$$= \frac{100 * W}{(100 - p_1)} * \frac{1}{\gamma_s} \text{ m}^3 = V_1$$

At moisture content of p (percent), similarly,

Vol. of sludge produced (V)

$$V = \frac{100 * W}{(100 - p)} * \frac{1}{\gamma_s} \text{ m}^3$$

From equation (i),

$$W = \frac{(100 - p_1) * V_1 * \gamma_s}{100}$$

From equation (ii),

$$W = \frac{(100 - p) * V * \gamma_s}{100}$$

Equating (iii) and (iv),

$$\frac{(100 - p_1) * V_1 * \gamma_s}{100} = \frac{(100 - p) * V * \gamma_s}{100}$$

$$V = V_1 \frac{(100 - p_1)}{(100 - p)} \qquad 6.1$$

Source of Sludge

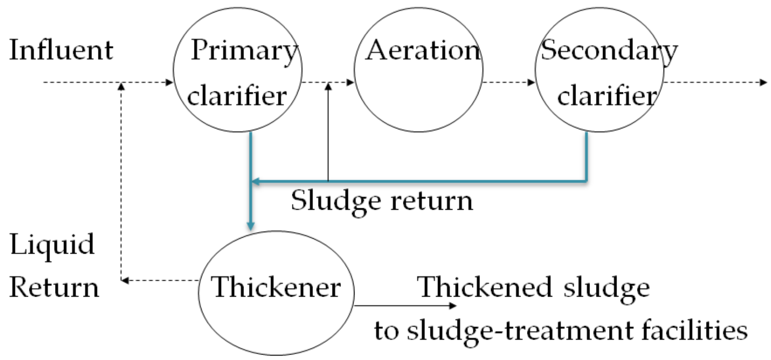


Figure 6-1: Sources of sludge

6.2 Sludge Processing and Disposal Methods

The principal methods used for sludge processing and disposal are reported in the Table 6-1.

Table 6-1: Sludge processing and disposal methods

Processing, disposal Function	Unit operation, unit process, or treatment method
Preliminary operations	Sludge pumping and grinding, Sludge blending and storage
Thickening	Gravity thickening, Flotation thickening, Centrifugation, Classification
Stabilization	Chlorine oxidation, Lime stabilization, Anaerobic digestion, Aerobic digestion, Heat treatment
Disinfection	Disinfection

Conditioning	Chemical conditioning
Dewatering	Centrifuge, Vacuum filter, Pressure filter, Horizontal belt filter, Drying bed, Lagoon
Drying	Dryer
Composting	Composting, Co-composting
Ultimate disposal	Landfill, Land application

i. Sludge Thickening

The purposes of sludge thickening are to reduce the sludge volume to be handled in the subsequent sludge processing units (pump, digester, and dewatering equipment) and to reduce the construction and operating costs of subsequent processes.

Sludge thickening is a procedure used to remove water and increase the solids content.

For example:

If waste activated sludge with 0.6 percent solids is thickened to a content of 3.0 percent solids, a five-fold decrease in sludge volume is achieved.

ii. Sludge Digestion Process

As pointed out earlier, the sludge withdrawn from the sedimentation basins contains a lot of putrescible organic matter, and if disposed of without any treatment, the organic matter may decompose, producing foul gases and a lot of nuisance, pollution and health hazards. In order to avoid such pollutions, the sludge is, first of all, stabilized by decomposing the organic matter under controlled **anaerobic conditions**, and then disposed of suitably after drying on drying beds, etc. The process of stabilization is called the sludge digestion; and the tank where the process is carried out is called the sludge digestion tank. In a sludge digestion process, the sludge gets broken into the following three forms:

(i) Digested sludge

It is a stable humus like solid matter, black in color, and with reduced moisture content, and, is therefore, having reduced volume (about $\frac{1}{3}$ times the undigested sludge volume). Moreover, the quality of digested sludge is much better than that of the undigested sludge, and it is free of pathogenic bacteria which are killed in the digestion process. It may still, however, contain cysts and eggs of bacteria, protozoa and worms.

(ii) Supernatant liquor

It includes the liquefied and finely divided solid matter, and is having high BOD (about 3000ppm).

(iii) Gases of decomposition

Gases like methane (65 to 70%), carbon dioxide (30%), and traces of other inert gases like nitrogen, hydrogen sulphide, etc. are evolved. They may be collected (particularly the methane which has a high calorific value) and used as a fuel.

The sludge gas, having 70% methane, has a fuel value of about 5800 kilo calorie/cu.m (i.e. 650 Btu per cu. ft. app.). The amount of gas produced, on an average, is about 0.9m^3 per kg of volatile solids reduced in the digestion. The gas produced thus varies with the sewage produced, and works out to about 14 to 18 liters per capita per day (usually 17l/c/d).

The digested sludge is dewatered, dried up, and used as fertilizer; while the gases produced are also used for fuel or for driving gas engines.

6.3 Stages in the Sludge Digestion Process

Three distinct stages have been found to occur in the biological action involved in the natural process of sludge digestion. These stages are:

- (i) Acid fermentation
- (ii) Acid regression
- (iii) Alkaline fermentation

Factors Affecting Sludge Digestion and Their Control

The important factors which affect the process of sludge digestion, and are, therefore, controlled in a digestion tank, are:

1. Temperature
2. pH value
3. Seeding with digested sludge
4. Mixing and stirring of the raw sludge with digested sludge.

Sludge Digestion Tank or Digesters

Constructional Details

A typical sludge digestion tank is shown in Figure 6-2:. It consists of a circular RC.C tank with hoppered bottom, and having a fixed or a floating type of roof over its top. The raw sludge is pumped into the tank, and when the tank is first put into operation, it is seeded with the digested sludge from another tank, as pointed out earlier. A screw pump with an arrangement for circulating the sludge from bottom to top of the tank or vice versa (by reversing the direction of rotation of the screw) is commonly used, for stirring the sludge. Sometimes, power driven mechanical devices may be used for stirring the sludge, although these are not very popular, at present.

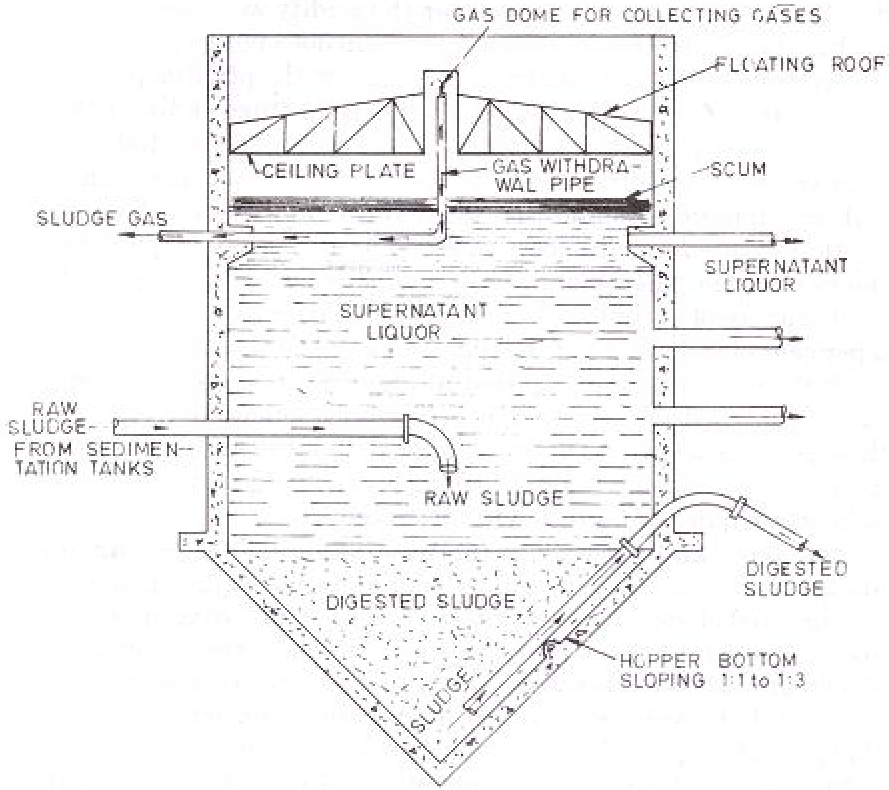


Figure 6-2: Cross section of a typical sludge digestion tanks

In cold countries, the tank may have to be provided with heating coils through which hot water is circulated in order that the temperature inside the tank is maintained at optimum digestion temperature level.

The gases of decomposition (chiefly methane and carbon dioxide) are collected in a gas dome (in smaller tanks) or collected separately in gas holders (in larger tanks) for subsequent use. The digested sludge which settles down to the hoppered bottom of the tank is removed under hydrostatic pressure, periodically, once a week or so. The supernatant liquor lying between the sludge and the scum is removed at suitable elevations, through a number of withdrawal pipes, as shown. The supernatant liquor, being higher in BOD and suspended solids contents, is sent back for treatment along with the raw sewage in the treatment plant. The scum formed at the top surface of the supernatant liquor is broken by the re-circulating flow or through the mechanical rakers called scum breakers.

Design Considerations

The digestion tanks are cylindrical shaped tanks (i.e. circular in plan) with diameter ranging between 3 to 12m. The bottom hoppered floor of the tank is given a slope of about 1:1 to 1:3 (i.e. 1 H: 3 V). However, when the sludge is moved to the outlet by means of some mechanical equipment, the bottom slopes may be made relatively flat.

The depth of the digestion tank is usually kept at about 6m or so.

Deeper tanks are costlier, though more effective. Except in very large plants, it is usual not to provide more than 2 units.

The capacity of the digestion tank is a function of sludge production, digestion period, degree of digestion required, loss of moisture, and conversion of organic matter. If the progress of sludge digestion is assumed to be linear, then the capacity of the digestion tank is given as:

$$V = \left(\frac{V_1 + V_2}{2}\right) * t \qquad 6.2$$

Where, V = Vol. of the digestion, m^3

V_1 = Raw sludge added per day, m^3/d

V_2 = Equivalent digested produced per day on completion of digestion, m^3/d

$\approx \frac{V_1}{3}$

t = Digestion period, d

When the daily digested sludge could not be removed (even though digestion gets completed) due to the factors, such as monsoon season, winter season, etc. ; then separate capacity for its storage should be provided in the tank. This capacity eventually amounts to V_2*T. Where, T is the number of days for which the digested sludge is stored, and is called ***monsoon storage***. The total digester volume is then given as:

$$V = \left(\frac{V_1 + V_2}{2}\right) * t + V_2 * T \qquad 6.3$$

However, when the change during digestion is assumed to be parabolic rather than linear, the average volume of digesting sludge will be $[\,V_1 - \frac{2}{3}(V_1 - V_2)\,]$ and the required capacity will be given by:

$$V = \left[V_1 - \frac{2}{3}(V_1 - V_2)\right] * t \qquad 6.4$$

- without monsoon storage

$$V = \left[V_1 - \frac{2}{3}(V_1 - V_2)\right] * t + V_2 * T \qquad 6.5$$

- with monsoon storage

The capacity of the sludge digestion tank calculated above may also be modified for the following additional factors:

(i) amount of sludge withdrawn and its interval
(ii) provision of adequate free-board at top
(iii) storage for winter
(iv) type of pre-treatment given to fresh sludge
(v) collection of gas; etc.

Depending upon all these variable factors, digestion tanks are also designed on per capita basis. The capacity provided per capita may range between 21 to 61 liters per capita (taking into account the usual one month digestion period) as shown in Table 6-2:.

Table 6-2: Design Capacities of the Digestion Tanks on Per Capita Basis

S. No.	Type of raw sludge to be digested	kg of volatile solids present per m^3 of sludge per month	Per capita capacity in m^3/capita
1	Primary sludge	8	0.021
2	Mixture of primary sludge and secondary sludge from trickling filters (humus tanks)	7.36	0.036
3	Mixture of primary sludge and secondary activated sludge	5.76	0.061
4	Chemically coagulated sludge	-	0.056

As pointed out earlier, the amount of the sludge gas produced in the digestion tank, ranges between 14 to 28 liters per capita per day (usually 17 liter per capita per day or 900 liters per gm of volatile solids digested, is quite common). The gas collected may be utilized for operating gas engines, and for heating sludge to promote quick digestion. However, the gas collection and its utilization is found fruitful only in ease of large plants serving more than 50,000 persons or so, so as to produce at least about 800 – 1000m^3 of gas per day.

Example 6-3

Design a digestion tank for the primary sludge with the help of following data:

(i) Average flow = $20*10^6$l/d
(ii) Total suspended solids in raw sewage = 300mg/l
(iii) Moisture content of digested sludge = 85%

Assume any other suitable data you require.

Solution:

Average sewage flow = $20*10^6$l/d

Total suspended solids = 300mg/l

Mass of suspended solids in $20*10^6$ liter of sewage flowing per day

$$= \frac{300 * 20 * 10^6}{10^6} \text{ kg} = 6000 \text{kg/day}$$

Assuming that 65% solids are removed in primary settling tanks,

Wt. of solids removed in the primary settling tank

$$= 65\% * 6000 \text{ kg/day} = 3900 \text{ kg/day}$$

Assuming that the fresh sludge has moisture content of 95%,

5 kg of dry solids will make = 100 kg of wet sludge and

3900 kg of dry solids will make

$$= \frac{100}{5} * 3900 \text{ kg of wet sludge per day} = 78{,}000 \text{ kg of wet sludge per day}$$

Assuming the sp. gravity of wet sludge as 1.02; i.e. Density = $1020 kg/m^3$,

The volume of raw sludge produced/day

$$= V_1 = \frac{78000}{1020} m^3/\text{day} = 76.47 m^3/\text{day}$$

The volume of the digested sludge (V_2) at 85% moisture content is given by equation 6.1 as:

$$V_2 = V_1 \frac{(100 - p_1)}{(100 - p)}$$

$$V_3 = V_1 \frac{(100 - 95)}{(100 - 85)}$$

$$V_2 = \frac{1}{3} V_1 = \frac{1}{3} * 76.47 = 25.49 \text{ m}^3/\text{day}$$

Assuming the digestion period as 30 days, we have the capacity of the required digestion tank, given by Eqn. 6.4 as:

$$\text{Capacity} = V = \left[V_1 - \frac{2}{3}(V_1 - V_2)\right] * t$$

$$= \left[76.47 - \frac{2}{3}(76.47 - 25.49)\right] * 30 = 1274.5 \text{ m}^3$$

Providing 6.0 m depth of the cylindrical digestion tank, we have

Cross-sectional area of the tank

$$= \frac{1274.5}{6} = 212.5 m^2$$

Diameter of tank

$$= \sqrt{\frac{212.5}{\frac{\pi}{4}}} = 16.45 \text{ m} \approx 16.5 \text{ m}$$

Hence, provide a cylindrical sludge digestion tank (typical section shown in Figure 6-2) 6m deep and 16.5m diameter, with an additional hoppered bottom of 1:1 slope for collection of digested sludge.

Example 6-4

A wastewater treatment plant produces 1000kg of dry solids per day at a moisture content of 95%. The solids are 70% volatile with a specific gravity of 1.05 and the remaining is non-volatile with specific gravity of 2.5. Find the sludge volume after digestion which reduces volatile solid content by 50% and decreases the moisture content by 90%.

Solution:

Total solids produced = 1000 kg (dry mass)

Volatile solids = 70% total solids = 700kg

Non-volatile solids = 30% TS = 300kg

Volatile solids removed in digestion

$$= 50\% \text{ volatile solids} = 50\% * 700\text{kg} = 350\text{kg}$$

Volatile solids left in digested sludge = 350kg

Non-volatile solids in digested sludge

$$= 300\text{kg (as in original sludge)}$$

Mass of water in wet digested sludge = 90%

10% mass of solids

$$= 300 + 350 = 650\text{kg}$$

10kg of solids make

$$= 100\text{kg of wet sludge}$$

Therefore, 10kg of solids contain

$$= 90 \text{ kg of water}$$

Or 650 kg of solids contain

$$= \frac{90}{10} * 650 = 5850\text{kg}$$

Density of volatile solids

$$= 1000\text{kg/m}^3 * 1.05\ (p_w * S_s) = 1050\text{kg/m}^3$$

Similarly density of non-volatile solids

$$= 1000 * 2.5 = 2500\text{kg/m}^3$$

Volume of volatile solids in wet sludge

$$= \frac{350}{1050} = 0.333\text{m}^3$$

Volume of non-volatile solids in wet sludge

$$= \frac{300}{2500} = 0.12m^3$$

Volume of water in wet sludge

$$= \frac{5850}{1000} = 5.85m^3$$

Hence, total volume of wet sludge

$$= 0.338 + 0.12 + 5.85 = 0.303m^3$$

Estimated Gas Production

If it is possible to analyze the wastewater to be treated, and to determine the characteristics of the sludge, its gas producing ability may be estimated. When such data cannot be obtained, the following approximate values may be used as a basis for estimating the amount of gas produced by digestion:

About 60% of the suspended solids of sewage are removed by sedimentation; 75% by chemical coagulation and settling; and 90% by complete treatment, such as by the activated sludge or the trickling filters, preceded and followed by sedimentation.

About 70% of the suspended solids in the sewage are volatile, and the reduction of the volatile matter in sludge, is about 65%. In digestion, the amount of gas produced is about 0.6cu. m per kg of volatile matter present in the sludge, or is about 0.9cu-m per kg of volatile matter reduced. The gas produced usually contains 65% methane, 30% carbon dioxide, and trace amounts of other gases. The heat content of methane is approximately 36000kJ/m^3 (8800kC/m^3).

Two Stage Digestion

While treating sewage on a large scale, two stage digestion of sludge is generally adopted instead of a single stage digestion, as discussed above. In two stage digestion, two digestion tanks, called primary and secondary digesters, are used.

Sludge is, first, admitted into the primary digestion tank (or primary digester) and is kept there for a period of about 7 to 10 days. The gas produced is collected in this unit. The partly digested sludge and supernatant liquor from this primary digester are then transferred to the secondary digestion tank (or secondary digester), where they are kept for a period of about 20 days or so. The digested sludge and supernatant liquor from the secondary digester are then finally removed, and disposed of suitably in a sanitary manner.

Two stage digestion gives the following advantages compared to single stage digestion:

(i) Two stage digestion is an effective method of preventing (or reducing greatly) any tendency for the sludge to short-circuit, as may happen in a single stage digester. The quality of supernatant liquor produced is, therefore, much better here.

(ii) Only the primary digester is provided with heating, stirring, gas collection arrangements; while the secondary unit merely acts as a closed settling tank, so as to produce clear super-natant liquor, thus affecting economy in construction.

(iii) Two stage digestion offers the freedom from large scum formations in any of the digestion tanks.

(iv) It has been estimated that the total cost with two stage digestion may be less than that for two tanks operated in parallel.

6.4 Disposal of Digested Sludge

The digested sludge from the digestion tank contains a lot of water, and is, therefore, first of all, dewatered or dried up, before further disposal either by burning or dumping. These methods of dewatering the sludge are discussed below:

Dewatering, Drying and Disposal of Sludge by Sludge Drying Beds

Drying of the digested sludge on open beds of land (called sludge drying beds) is quite suitable for hot countries.

Sludge drying beds are open beds of land, 45 to 60cm deep, and consisting of about 30 to 45cm thick graded layers of gravel or crushed stone varying in size from 15 cm at bottom to 1.25cm at top, and overlain by 10 to 15cm thick coarse sand layer. Open jointed under-drain pipes (15cm in diameter) @ 5 to 7m c/c spacing are laid below the grave layer in valleys, as shown in Figure 6-3: Sludge drying beds, at a longitudinal slope of about 1 in 100. The beds are about 15 x 30m in plan, and are surrounded by brick walls rising about 1 meter above the sand surface, as shown.

The sewage sludge from the digestion tank is brought and spread over the top of the drying beds to a depth of about 20 to 30cm, through distribution troughs having openings of about 15 cm x 20 cm at a distances of about 2m or so.

A portion of the moisture drains through the bed, while most of it is evaporated to the atmosphere. It usually takes about two weeks to two months, for drying the sludge, depending on the weather and condition of the bed.

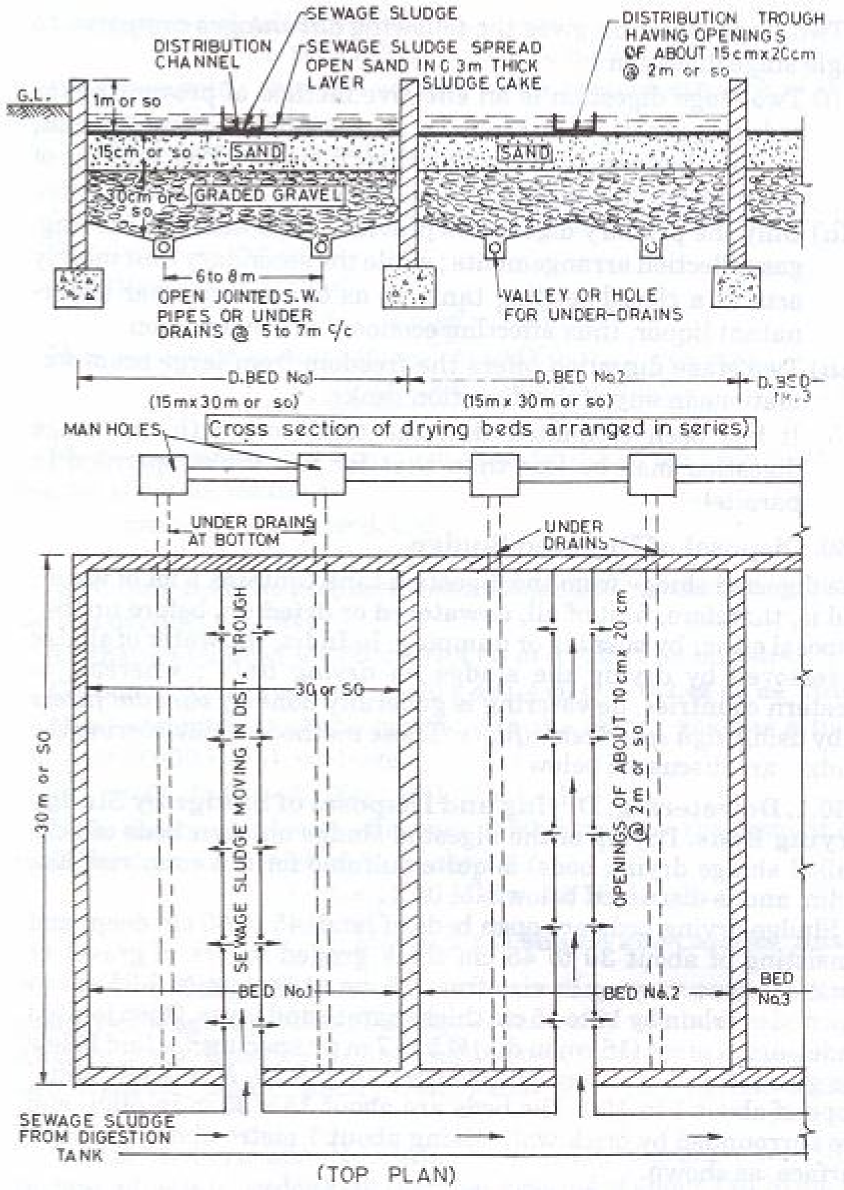


Figure 6-3: Sludge drying beds

The required area for sludge drying beds normally ranges between 0.05 to 0.2 sq. m per capita, as shown in Table 1-1. In some instances, the required area is reduced by building a glass roof over the beds, to give protection against rain and snow.

Table 6-3 Approximate area of Drying beds for different types of sludge

S. No.	Source of sludge	Area in m^2/capita	
		Open beds	Covered beds
1	Primary clarifiers	0.1	0.08
2	Intermittent sand filters	0.1	0.08
3	Standard rate trickling filters	0.12	0.10
4	High rate trickling filters	0.15	0.12
5	Activated sludge	0.18	0.14
6	Coagulated sludge	0.20	0.15

Sludge should never be applied to a bed until the preceding dose has been removed. Hence, several drying beds will generally be required, with their number increasing with an increase in the number of days for which the sludge is kept on the beds. Normally, sludge is removed from the beds after a period of about 7 - 10 days; as within this period, about 30% of the moisture goes away and the surface of sludge gets cracked. The sludge cakes are then removed by spades, and they are dumped into a pit for further drying. The dried sludge is generally used as manure in our country, as it contains 1.7% nitrogen, 1.5% phosphoric acid, and 0.5% potash. It may also be used for filling up low lying areas. It may sometimes be disposed of by burning (i.e. by incineration).

Mechanical Methods of Dewatering Sludge

In developed countries, digested sludge (sometimes even raw sludge), is often dried or dewatered by mechanical means, such as by vacuum filtration or by high speed centrifuges.

In vacuum filtration process, the sludge is first mixed with a coagulant such as ferric chloride, and then conveyed to a vacuum filter, consisting of a hollow rotating drum, covered with a replaceable filter cloth. The drum rotates partly submerging into the sludge. The vacuum created by a pump within the drum draws the moisture from the sludge through the cloth. The sludge cake which is formed on the outside of the drum is removed by a scraper, as the drum rotates.

High-speed centrifuges are also used for drying of raw or digested sludge, and are becoming more popular because of small area requirements. These methods may remove about 50% moisture.

Vacuum filtration or centrifugation of raw sludge is often adopted in situation where sludge is to be disposed of by incineration *(i.e.* burning). These mechanical methods of drying are generally used when the available area is less than that required for sludge dryidge beds, or where the climates are too cold or at places where rains are frequent as not to permit natural drying, or as a preliminary to heat drying for making fertilizer.

Elutriation of sludge

Before the sludge is dewatered by vacuum filtration method, it is generally elutriated. Elutriation is the process of washing the sludge water, to remove the organic and fatty acids from it. Sometimes, before dewatering, the sludge is thickened or conditioned, and then elutriated.
During the sludge digestion process, the volatile acids, alcohols, and organic acids are formed, which if not removed, will interfere with coagulation process during dewatering by vacuum filtration. If elutriation is done before dewatering, it will reduce the quantity of coagulants considerably.
Sludge elutriation is carried out in sludge or multiple tanks by washing the sludge with water. During washing, the solids are continuously kept in suspension by air or by mechanical.
Single stage, or multistage, or counter-current washing may be employed as one of the three-methods for washing. The choice of any of these methods will depend upon the availability of water, because single stage elutriation will require for the same alkalinity reduction, 2.5 times the water required for two-stage elutriation, and 5 times that required for counter-current washing.
In the elutriation process, sludge and water are mixed in a chamber fitted with mechanical devices, keeping it for 20 seconds (i.e. detention period). The sludge is then settled in settling tanks, and excess water is decanted. The maximum surface loading on settling tank may be 40m3/m2/day with a detention period of 4 hours.
Counter-current elutriation is generally carried out in twin tanks, similar to the sedimentation tanks, in which sludge and water enter at opposite ends. Piping and channels are so provided that wash water entering the second stage tank comes first in contact with sludge already washed in the first stage. The quantity of wash water required is about 2 to 3 times the quantity of sludge elutriated.

6.5 Disposal of Dewatered Sludge

The dewatered sludge obtained from mechanical devices of western countries, is generally heat dried, so as to produce fertilizers, As a matter of fact, the mechanical dewatering removes only about 50% of the moisture, and hence the mechanically dewatered sludge is actually heated, so as to fully remove the moisture from it. The dry residue is used as manure. This method is looked upon more as a method of producing fertilizer rather than as a method of sludge disposal, because if this method is adopted only for sludge disposal, it proves to be extremely costly, and thus feasible only for rich countries.

The wet sludge, after mechanical dewatering, is sometimes, directly disposed of either in sea or in underground trenches, or burnt, as discussed below:

Disposal by Dumping into the Sea

The dewatered wet sludge may, sometimes, be discharged at sea from hopper barges or through outfall sewers. This method can, however, be adopted only in case of cities situated on sea shores, and where the direction of the normal winds are such as to take the discharged sludge into the sea, away from the shore line.

Disposal by Burial into the Trenches

In this method, the digested sludge, without dewatering is run into trenches, which are 0.9m wide * 0.6m deep and rectangular spaced at 1 to 1.5m apart in parallel rows. When the sludge has dried to a firm state, it is covered at top with a thin layer of soil. After about a month, the land is ploughed up with powdered lime and planted with crops.

Disposal by Incineration

The dewatered wet sludge produced in waste water treatment plant may also be disposed of by burning, in suitably designed incinerators, when sufficient space is not available for its burial near the plant site, or the sludge cannot be dried and used as manure. The following types of incinerators (furnaces) have primarily been designed and used for incinerating wet sludge:

(1) Multiple-hearth furnace
(2) Fluid-bed furnace
(3) Flash-type of furnace
(4) Infra-red (Electric or Radiant heat) furnace.

Use of Lagoons for Disposal of Raw Sludge

This method is, sometimes, used at smaller places for disposing of raw sludge without digestion. In this method, the raw sludge is kept at rest in a large shallow open pond, called a *lagoon.* The detention period is l to 2 months, and may extend up to 6 months. During its detention in the lagoon, the sludge undergoes anaerobic digestion thereby getting stabilized. Due to this anaerobic decomposition of sludge, foul gases will be evolved from a lagoon; and hence the lagoons should be located away from the town, and direction of the common winds should be such that the smells are not carried towards any localities.

A typical section for a lagoon is shown in Figure 6-4:. It is a shallow pit, 0.6 to 1.2m deep, formed by excavating the ground.

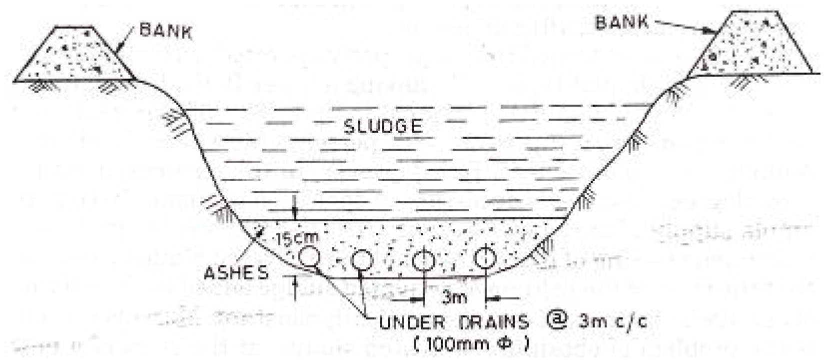


Figure 6-4: Typical section of a lagoon for disposal of raw sludge

At the bottom of this pit, a 15 cm thick layer of ashes or clinker is placed. Agricultural tile drains of about 10 cm diameter are laid at bottom as *under-drains.* These are placed at about 3m centre to centre spacing. Banks are formed on both sides of the pit from the excavated earth as shown.

After the sludge has been stabilized, and the moisture has been drained away or evaporated during its detention-fin the lagoon, the contents are dug out to about half of their original volume, and used as manure.

This method of sludge disposal is quite cheap (as no digestion tanks are required), but the greatest drawback is the evolution and eruption of foul gases, polluting the environment. Its use is, therefore, restricted only to non residential areas.

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