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Chemical Engineering Department

*“Design of a Domestic Wastewater Treatment
Plant for a Residential Community”*

*This project submitted for the partial fulfillment of Bachelors
of Science degree in chemical engineering department*

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شكر وتقدير

الحمد لله الذي لا يبلغ مدحه المادحون، ولا يحصى نعمه العادون، ولا يؤدي حقه المجتهدون، وصل اللهم على أشرف المرسلين، سيدنا وحبيبنا محمد - صلى الله عليه وسلم-. نود أن نتقدم بجزيل الشكر والامتنان، إلى أسرة قسم الهندسة الكيميائية من أعضاء هيئة تدريس، ومهندسين، وموظفين، وفنيين، ونخص منهم بالذكر، مشرفنا، وأستاذنا الفاضل الدكتور "محمد علي البهليل"، لما أبداه من جهود عظيمة، وتوجيهات سديدة، ومتابعة دقيقة، في جميع مراحل هذا البحث، حتى توجت بهذه النتائج. ونخص بالشكر، والتقدير أيضاً، "إدارة منظومة معالجة مياه الصرف الصحي بمنطقة الهضبة" من مهندسين، وفنيين، وعاملين، على إتاحة الفرصة لزيارة الموقع، وعلى الشروح الوافية لكل مراحل المعالجة. وأخيراً، لكل من ساندنا ولو بكلمة طيبة، نقدم لهم شكرنا، وامتناننا، وتقديرنا، والله الموفق.

عائشة عادل سالم شاقان.

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Aesha Adel S. Shaghan.

Hebah Khalifa M. Raowe.

Dedication

To our mothers,

To our fathers,

To all our families,

To all our friends,

To our project supervisor,

To the wastewater treatment community,

*I dedicate this project hoping that someone will get it
beneficial and helpful.*

Aisha Adel S. Shaghan & Heba Khalifa M. Rewei.

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List of Symbols

	<i>Definition</i>	<i>Unit</i>
hl	Head loss through the rack	M
g	Gravity Acceleration	m/s²
b	Minimum clear spacing of bar	Mm
τ	Detention time	Min
W	Width	M
L	Length	M
BOD	Biochemical oxygen demand	mg/L
TSS	Total suspended solids	mg/L
D	Depth	M
M	Max flow	m³/sec
kd	Endogenous decay coefficient	d⁻¹
V	Volume	m³
θ_c	Mean cell residence time based on solids	Day
Q	Influent wastewater flow rate	m³/d
Y	Yield coefficient over finite period of log growth	g/g
S_o	Influent soluble BOD ₅ concentration	mg/L
S	Effluent soluble BOD ₅ concentration	mg/L
Q_r	Waste sludge flow rate from the sludge return line	m³/d
Q_w	Waste sludge flow rate from aeration tank	m³/d
Q_e	Flow of treated effluent	m³/d
Sc	Secondary clarifiers	m³/sec
K	First order susceptibility coefficient	min⁻¹

Project Summary

This project outlines a complete design of a domestic wastewater treatment plant. This WWTP intended to serve a medium population city of 75,000 inhabitants. The treated effluent wastewater will in compliance with the local and international standards. The treated effluent wastewater will pathogenic free and can be safely used for irrigation or discharged in the sea without any risk. The average share of wastewater per person assumed to be 620 Liters per day. The design calculations of the domestic wastewater treatment plant (WWTP) conducted to the four essential treatment operations namely preliminary, primary, secondary and tertiary treatment. Design calculations conducted to each facility of the above-mentioned treatments. Results found to be comparable with what presented in the literature and several working wastewater treatment plants.

The impact of important factors on the design investigated. These factors include the population increase, type of domestic wastewater weak, moderate, and strong wastewater represented by the influent BOD and Total suspended solids. Finally, the impact of the kinetic and growth parameters on the design and size of the various facilities in the wastewater treatment plant (WWTP).

Chapter One

Introduction

1.1- Introduction:

Wastewater can be defined as the flow of used water discharged, from homes, businesses industries, commercial activities and institutions, which is direct treatment plants by a carefully designed and engineered network of pipes.

1.1.1 Sources of Wastewater.

Following are the principal sources of wastewater

- Domestic
It is the wastewater from houses, offices, other buildings, hotels and institutions.
- Industrial:
It is the liquid waste from the industrial places from their different Industrial processes like dying, paper matting, tanneries, chemical industries, etc.
- Storm Water:
It includes surface runoff generated from rainfall and the street wash.

Wastewater treatment is the process of purification of wastewater from impurities, suspended substances, pollutants and organic matter to become reusable (non-human) or to be disposed of in the waterways without causing contamination.

Sewage treatment involves several physical, chemical and biological stages. Sewage treatment involves several physical, chemical and biological phases if, for example, sewage discharged without treatment by dumping it at sea or river, the following will occur:

The pathogenic microbes that spread to humans spread through bathing or drinking. Microbes analyze dissolved oxygen depleting organic matter in water to increase the demand for bio-oxygen, leading to the death of aquatic organisms such as fish and crustaceans. The activation of anaerobic microbes due to exhaustion of

dissolved oxygen and fermentation of organic matter can cause sludge fouling and stinky odors.

Municipal wastewater contains over 99.9 percent water; the remaining materials include suspended and dissolved organic and inorganic matter as well as microorganisms. These materials give physical, chemical and biological qualities that are characteristics of residential and industrial wastewaters [1].

Factors affecting water withdrawal rates are:

- 1) Climate.
- 2) Geographic location.
- 3) Size and economic conditions of the community.
- 4) Degree of industrialization.
- 5) Metered water supply.
- 6) Cost of water and supply.

1.1.2 Wastewater Components and Analysis.

The characteristics of wastewater are broadly classified into physical, chemical, and biological according to the type of measurement test that has performed. The analyses range from the very specific quantitative tests usually applied for chemicals to the broad group tests applied to biological classes.

The ranges commonly found in domestic wastewater for some of the common components and their degree of removal in conventional primary-secondary plants. Although the nitrogen, phosphorus and dissolved solids can removed by the addition of chemicals and certain tertiary methods, they are not easily removed in a conventional plant. If water reuse is to be widely practiced, these minerals will removed.

1.2 Characteristics of Domestic Wastewater.

1.2.1 Physical Characteristics.

Fresh Aerobic, domestic wastewater has said to have the odor of kerosene or freshly turned earth. Aged septic sewage is considerably more offensive to the olfactory nerves. The characteristic rotten egg odor of hydrogen sulfide and the mercaptans is indicative of septic sewage. Fresh sewage is typically gray in color. Septic sewage is black. Wastewater temperature normally range between 10 and 20°C. In general, the temperature of the wastewater will be higher than that of the water supply. This is because of the addition of warm from households and heating within the plumbing system of the structure.

One cubic meter of wastewater weighs approximately 1,000,000 grams. It will contain about 500 grams of solids. One-half of the solids will be dissolved solids such as calcium, sodium, and soluble organic compounds. The remaining 250 grams will be insoluble. The insoluble fraction consists of about 125 grams of material that will settle out of the liquid fraction in 30 minutes under quiescent condition. The remaining 125 grams will remain in suspension for a very long time. The result is that wastewater is highly turbid.

1.2.2 Chemical Characteristics.

The chemical oxygen demand (COD) test is used to determine the oxygen equivalent of the organic matter that can be oxidized by a strong chemical oxidized agent (potassium dichromate) in an acid medium. The COD of waste, in general, will be greater than the BOD_5 because more compounds can oxidized chemically than biologically, and because BOD_5 does not equal ultimate BOD.

The COD test can be conducted in about three hours. If it can be correlated with BOD₅, it can be used to aid in the operation and control of the wastewater treatment plant (WWTP).

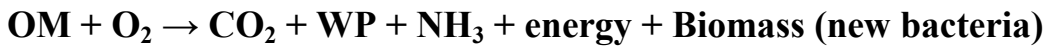
Total kjeldahl nitrogen (TKN) is a measure of the total organic and ammonia nitrogen in the wastewater. TKN gives a measure of the availability of nitrogen for building cells, as well as the potential nitrogenous oxygen demand that will have to be satisfied.

The chemical characteristics of wastewater can adversely affect the environment in many different ways. Soluble organics can deplete oxygen levels in streams, and give taste and odor to water supplies. Toxic materials can affect food chains as well as public health. Nutrients can cause eutrophication of lakes. Although some chemical tests are specific, many determine broad classification due to the variety of compounds found in wastewater.

- **Organic matter:**

The principal categories of biodegradable organic matter in wastewater are proteins, carbohydrates, and lipids. Proteins are high molecular weight compounds composed of amino acids. Carbohydrates are polyhydroxylated compounds, such as sugar, starches, and cellulose. Lipids refer to a variety of organic substances such as fats, oils, and grease. A typical domestic sewage contains 40-60 % proteins, 25-50 % carbohydrate, and 10% lipids.

There are several common tests for organic matter, all of which are non-specific and none of which give a complete determination. If organic matter is present in the wastewater, microorganisms in the wastewater or in the environment will oxidize the organic matter using molecular oxygen as the oxidizing agent. This transformation may be represented by:



OM: Organic matter WP: Waste products Biomass: New bacteria

Biochemical oxygen demand (BOD) used commonly as an indirect measure of organic matter. The BOD of a wastewater related to the quantity secondary treatment or by natural aeration of the receiving water.

The sample of wastewater seeded with bacteria and nutrients if necessary, and incubated at 20°C for 5 days, although other periods and temperatures can used. After incubation, the change in the dissolved oxygen concentration in the sample measured and five day BOD (BOD₅) expressed as mg/O₂ liter of wastewater.

- **Nitrogen:**

Both nitrogen and phosphorus are receiving increased attention because their compounds promote unwanted growth of algae and aquatic plants.

- **Phosphorus:**

Phosphorus enters the wastewater through human wastes, primarily urine, and through Phosphate compounds used as builders in detergent formulation. Each of these sources contributes significantly to the total amount of phosphorus.

- **Gases:**

Of all the gasses present in the water, Oxygen is the most important. Dissolved oxygen is necessary for the respiration of aerobic microorganisms and other life forms. If oxygen is at too low a level, noxious odors result because the carbon waste products become methane instead of carbon dioxide, the sulfur becomes hydrogen sulfide instead of sulfates, and the nitrogen remains as ammonia or possibly oxidized to nitrite.

1.2.3 Biological Characteristics.

Biological tests on water and wastewater determine whether pathogenic organisms are present by testing for certain indicator organisms. Biological information needed to measure water quality for such uses as drinking and swimming, and to assess the degree of treatment of the wastewater before its discharge to the environment.

1.2.4 Wastewater Microbiology.

- Role of Microorganisms.

The stabilization of organic matter accomplished biologically using a variety of microorganisms. The microorganisms 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 by gravity settling.

It is important to note that unless the cell tissue produced from the organic matter is removed from the solution, complete treatment will not be accomplished because the cell tissue, which itself is organic, will be measured as BOD in the effluent. If the cell tissue is not removed, the only treatment that will be achieved is that associated with the bacterial conversion of a portion of the organic matter originally present to various gaseous final products.

Some Microbes of Interest in Wastewater Treatment :

- 1) Bacteria
- 2) Fungi
- 3) Algae
- 4) Protozoa

- Bacteria.

The highest population of microorganisms in a wastewater treatment plant will belong to the bacteria. They are single-celled protists, which use soluble food.

Condition in the treatment plant are adjusted so that chemoheterotrophs Predominate.

1.2.5 Bacterial Biochemistry.**- Metabolism:**

The general term that describes all of the chemical activities performed by a cell is metabolism. This in turn divided into two parts: Catabolism and anabolism.

Catabolism includes all the biochemical processes by which a substrate degraded to end products with the release of energy. In wastewater treatment, the substrate oxidized. The oxidation process releases energy that transferred to an energy carrier, which stores it for future use by the bacterium. Anabolism includes all the biochemical processes by which the bacterium synthesizes new cells. The synthesis process driven by the energy that was stored in the energy carrier.

1.3 Decomposition of Waste:

The type of electron acceptor available for catabolism determines the type of decomposition (that is, aerobic, anoxic, or anaerobic) used by a mixed culture of microorganisms.

Each type of decomposition has peculiar characteristics, which affect its use in waste treatment.

1.3.1 Aerobic Decomposition:

From our discussion of bacterial metabolism, you will recall that molecular oxygen must be present as the terminal electron acceptor for decomposition to proceed by

aerobic oxidation. As in natural water bodies, the oxygen is measured as DO. When oxygen is present, it is the only terminal electron acceptor used. Hence, the chemical end products of decomposition are primarily carbon dioxide, water, and new cell material. Odiferous gaseous end products kept to a minimum. In healthy natural water systems, aerobic decomposition is the principal means of self-purification.

A wider spectrum of organic material can be oxidized aerobically than by any other type of decomposition. This fact, coupled with the fact that the final end products are oxidized to a very low energy level, results in a more stable end product (that is, one that can be disposed of without damage to the environment and without creating a nuisance condition) than can be achieved by the other oxidation systems.

Because of the large amount of energy released in aerobic oxidation, most aerobic organisms are capable of high growth rates. Consequently, in these systems, this means that more biological sludge is generated in aerobic oxidation than in the other oxidation systems.

Aerobic decomposition is the method of choice for large quantities of dilute wastewater because decomposition is rapid, efficient, and has a low odor potential. For high strength wastewater, it is not suitable because of the difficulty in supplying enough oxygen and because of the large amount of biological sludge that is produced. In small communities and in special industrial applications where aerated lagoons are used, wastewaters with BOD₅s up to 3000 mg/L may be treated satisfactorily by aerobic decomposition.

1.3.2 Anaerobic Decomposition:

In order to achieve anaerobic decomposition, molecular oxygen and nitrate must not be present as terminal electron acceptors. Sulfate, Carbon dioxide, and organic

compounds that can reduced serve as terminal electron acceptors. The reduction of sulfate results in the production of hydrogen sulfide and group of equally odiferous organic sulfur compounds called mercaptans. The anaerobic decomposition (fermentation of organic matter generally is considered to be a two-step process. In the first step, complex organic compounds fermented to low molecular weight fatty acids (volatile acids). In the second step, the organic acids converted to methane.

Carbon dioxide serves as the electron acceptor. Anaerobic decomposition yields carbon dioxide, methane, and water as the major end products. Additional end products include ammonia, hydrogen sulfide, and mercaptans. As consequence of these last three compounds, anaerobic decomposition characterized by an unbelievably horrid stench!

Because only small amounts of energy released during anaerobic oxidation, the amount of cell production is low. Thus, sludge production is low. We make use of this fact in wastewater treatment by using anaerobic decomposition to stabilize sludge produced during aerobic and anoxic decomposition.

Direct anaerobic decomposition of wastewater generally is not feasible for dilute waste. The optimum growth temperature for the anaerobic bacteria is at the upper end of the mesophilic range. Thus, to get reasonable biodegradation, we must elevate the temperature of the culture. For dilute wastewater, this is not digestion is quite appropriate [2].

1.4 The Main Objectives of This Project.

The main objectives of this work is to perform complete design calculations of the various operations in the domestic wastewater treatment plant (WWTP). This plant will serve an intermediate town with 75,000 inhabitants.

This design project hopefully will reach to these ultimate objectives:

- Reach acceptable levels for the disposed domestic wastewater.
- Achieve a safe level for re-use of the treated wastewater in a different field's e.g. agricultural irrigation, ground water recharge, cooling and other industrial purposes.
- Insure a sufficient degree of protection of health from adverse effects of pollution and disease transmission by controlling the quality of treated wastewater.
- Insure the safe application of the resulting dried sludge obtained from sludge treatment in agriculture purposes.
- Make full application of the resulting gases methane and carbon dioxide obtained from the anaerobic treatment process.

Chapter Two

Treatment Process

2.1 Wastewater Treatment.

Untreated sewage is dangerous to public health because it contributes to environmental water, land and air pollution. This facilitates transmission of disease, depreciation of land and degradation of the environment. The waste must thoroughly treated to produce an effluent that is safe and environmentally acceptable, which finally conveyed to disposal sites.

The fundamental equation for wastewater treatment is:

$$\text{Organic waste} + \text{oxygen} + \text{bacteria} = \text{Treated waste} = \text{New bacterial cells}$$

Discharging highly polluting waste into a body of water has negative effects on human, animal and plant life. There is a reduction of dissolved oxygen, which may affect aquatic life; at the same time, the absence of dissolved oxygen in the water leads to septic conditions. Health hazards to communities are increased and social amenities such as swimming decreased. Too many pollutants reduce the self-purification capacity of water, especially at the point of mixing, and they promote excessive growth of aquatic plants. Polluted waters are aesthetically objectionable because they emit unpleasant odors.

Wastewater should treated to the following:

- 1) Reduce environmental pollution, e.g., receiving water bodies and land.
- 2) Reduce costs of treatment of water supplies.
- 3) Render obnoxious materials, e.g., sludge, harmless.
- 4) Conform to any river standard in terms of suspended solids and BOD, i.e., to comply with official, designated standards.

- 5) Allow for recovery of useful by-products e.g., effluent for irrigation or dry sludge as a fertilizer.
- 6) Kill all pathogenic microorganisms.
- 7) Avoid any nuisance appearance.

The theory of sewage treatment plants is that microorganisms will metabolize the available organic matter. The design is to provide an apparatus or environmental conditions in which this action will take place as quickly, economically as possible. Wastewater treatment, therefore, should prevent pollution of water by matter, inorganic matter and toxic substances. It should further nuisances and minimize health hazards in the environment. Treatment should produce an effluent that is suitable for its intended re-use, and will not constitute a risk to public health. To achieve its aim, wastewater treatment must produce an effluent of certain quality or standard. The relevant authority should establish the required effluent quality.

The main objective of conventional wastewater treatment processes are reduction of biochemical oxygen demand, suspended solids, and pathogenic organisms. In addition, it may be necessary to remove nutrients, toxic components, non-biodegradable compounds, and dissolved solids. Since most contaminants are present in low concentration, the treatment processes must be able to function effectively with dilute streams.

Knowledge of the nature of the wastewater intended for treatment is very important issue. Waste varies in its characteristics. It is important to know the quantity of wastewater to be treat as well as the BOD load, suspended solids content and settle able solids content of the sewage. Wastewater can treated aerobically available.

There are three types of bacteria commonly used:

- 1) **Aerobic bacteria** - Those that only function in an environment where there is free oxygen or dissolved oxygen.
- 2) **Anaerobic bacteria** - Those that function well in an environment where there is combined oxygen, not dissolved oxygen.
- 3) **Facultative bacteria** - Those that can exist in both aerobic and anaerobic conditions.

All three types of bacteria may be present within a given treatment plant [3].

2.1.1 Municipal Wastewater Treatment Systems.

This discussion of municipal wastewater treatment systems follows the U.S. Environmental protection agency publication, environmental pollution control alternatives: Municipal wastewater.

The domestic municipal wastewater treatment must follow the following three major categories:

- 1) Preliminary treatment.
- 2) Primary treatment.
- 3) Secondary treatment.
- 4) Advanced treatment.

2.1.2 Preliminary Treatment.

a) Main Objectives.

The main aim in preliminary treatment is to protect the treatment works. This involves the removal of material such as grease and grit which may be adversely interfere with the waste water treatment.

In preliminary treatment the sewer load can carry six times the dry weather flow (DWF). Three times the DWF should be stored in overflow tanks; it

can be brought back to the system or disposed of after a detention time of 1 day. Excess (three DWF) of low strength sewage can be discharged into a river; however this should be avoided. There should be a screen to hold back a large particles entering the system. The grit removed before sedimentation.

A comminutor is used for breaking down large particles that might have gone through the screens.

b) Screens.

Screens are essential for the removal of large solids. Two types of screens are used in treatment plants: hand rake screen and mechanical rakes. For flows less than 200 cubic in per day, the hand rake screen is inclined at 30 to 60 degrees to the horizontal. The level of the waste water is lower after the screening due to head loss caused by trapped screenings. These are removed manually with hand rakes and must be properly disposed of in tertiary treatment.

Mechanical rakes present a concave face to the flow of wastewater. Screenings discharged into the trough and later carried away for disposal.

The velocity through the bars should be 0.6-1 m/s. higher velocity might force some large screenings through the bars. The bar spacing is normally 10-40 mm. Screenings are disposed of either by drying followed by burning or by burying (tertiary treatment), these are rich in organic matter and therefore should be handled with care.

A macerator used to cut screenings into smaller pieces; where desirable, the cut screenings can pass together with wastewater or be disposed of.

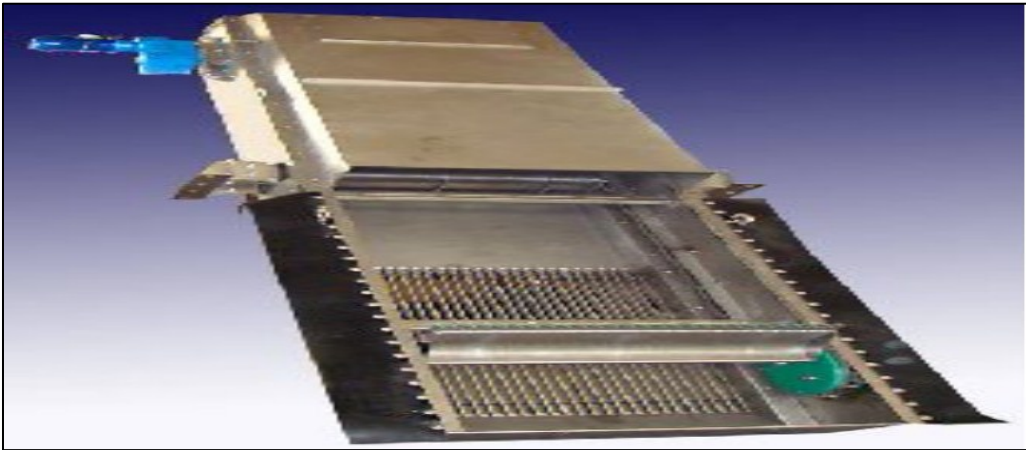


Figure (2.1): Bar Screen [1].

c) Grit Chambers.

Grit chambers are for removal of sand, gravel and similar debris. Grit chambers provided to protect moving equipment from abrasion and wear; they are located immediately after screens. The chambers are shallow, elongated channels that allow settlement of particles with specific gravity of 2.65 or more. Mechanical rakes can assist in the removal of grit that accumulates before the screen.

A flume is necessary for the measurement of daily flows. DWF defined as the average daily flow rate of waste on days when not more than 2.5 mm of rain has fallen in the previous 24 hours. The DWF is often equal to the water consumption minus the evaporation plus the infiltration or minus the exfiltration (leakage outwards). Infiltration may contribute up to 30 per cent of DWF. Inert dense material such as sand, broken glass, silt, and pebbles called grit.

If these materials not removed from the wastewater, they abrade pumps and other mechanical devices, causing undue wear. In addition, they have a tendency to settle in corners and bends, reducing flow capacity and, ultimately, clogging pipes and channels.

There are three basic types of grit removal devices:

- Velocity controlled grit chambers.
- Aerated grit chambers.
- Constant level short-term sedimentation basins.

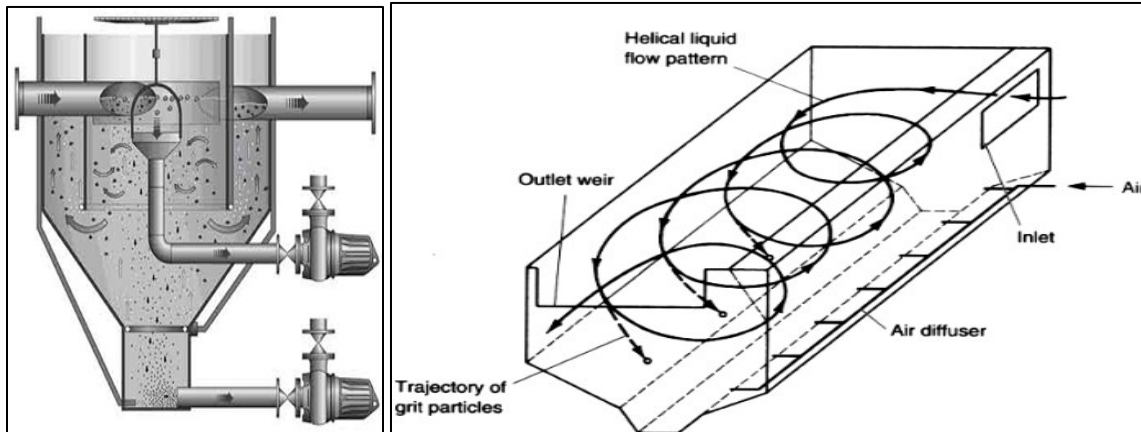


Figure (2.2): Aerated Grit Chambers. [1]

d) Oil and Grease in Wastewater.

Domestic and industrial sources are the chief contributors of grease in wastewater. Domestic examples include human faeces, kitchen waste and laundry waste. Industrial sources include slaughterhouses, tanneries, oil refineries, soap factories and garages.

Grease removal from wastewater is very important as it prevents oil “slick” which is a formation of an oil cover the wastewater. Oil slick is unsightly and may interfere with aquatic life. It will not allow entry of atmospheric oxygen into the wastewater. It may also cause coating of various components in sag treatment plants. In piping, grease causes reduction of diameters through coating, especially on bends. However, this depends on the amount of grease. Grease may cause coating of distributor and filter media in trickling filters. Clogging of air diffuses may occur with artificial aeration in lagoons. In receiving water bodies, grease

causes oil slick, which in turn affects aquatic life because it prevents aeration of water by atmospheric air. The presence of grease can deter effective wastewater treatment. Therefore, its removal is vital. Oil and grease in wastewater can be removed in two ways, namely, dissolved air flotation (DAF) or electrolysis.

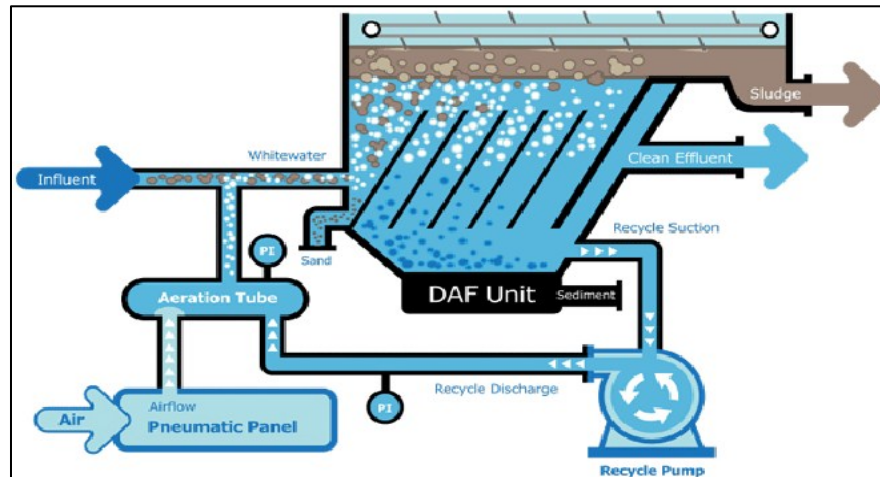


Figure (2.3): Dissolved Air Flotation System Unit for Oil and Grease Removal. [3]

Figure (2.4) shows a dissolved air flotation system (DAF), where air is introduced into the wastewater at very high pressure. Gas bubbles attach themselves to oil particles that rise to the top (buoyancy) forming scum. [3]

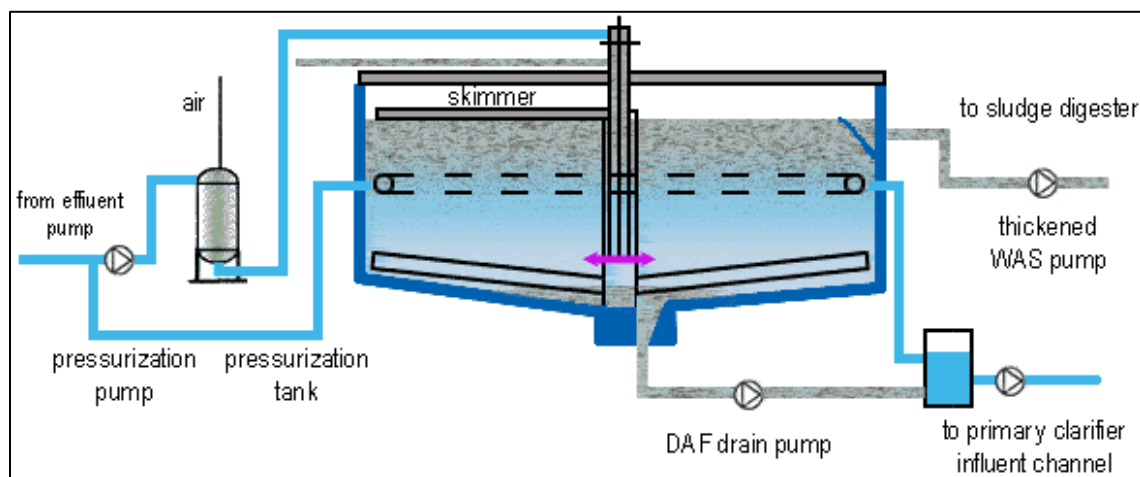


Figure (2.4): Dissolved Air Flotation System Unit for Sludge Thickening Facility [3].

The scum is then removed manually or mechanically. With the use of electrolysis, bubbles of gas are generated between positive and negative electrodes. Reduction of the apparent specific gravity of oil particles by attachment of gas bubbles occurs, and this causes the oil particles to rise to the top. The scum is then manually or mechanically removed [5].

2.2 Unit Operations of Pretreatment.

Several devices and structures are placed upstream of the primary treatment operation to provide protection to the wastewater treatment. The first device, at the most plants preliminary treatment used to protect pumping equipment and facilitate subsequent treatment processes. Preliminary devices designed to remove or cut up the larger suspended and floating solids, to remove the heavy inorganic solids, and to remove excessive amounts of oils or greases.

2.3 Unit Operation of Primary Treatment.

With the screening completed and the grit removed, the wastewater still contains light organic suspended solids, some of which can be removed from the sewage by gravity in a sedimentation tank. These tanks can be round or rectangular, are usually about 3.5 m deep, and hold the wastewater for periods of two to three hours. The mass of settled solids is called raw sludge. The sludge is removed from the sedimentation tank by mechanical scrapers and pumps. Floating materials, such as grease and oil, rise to the surface of the sedimentation tank, where they are collected by a surface skimming system and removed from the tank for further processing. The solid concentration is much lower in a primary tank than in a water-treatment plant clarifier. Thus, the overflow rates and weir loading rates differ appreciably. Overflow rates are commonly in the range 25 to 60 m³/d, the Great Lakes-Upper Mississippi River Board of State Sanitary Engineers (GLUMBRM) recommends

that weir loading (hydraulic flow over the effluent weir) rates not exceed $120\text{m}^3/d$ of flow per m of weir length ($\text{m}^3/d.m$). For plants with average flows less than 0.04meter cubic per second. For larger flows, the recommended rate is 190 meter cubic per day. As mentioned previously, approximately 50to 60 percent of the raw sewage suspended solid and as much as 30 to 35 percent of the raw sewage BOD_5 may be remove in the primary tank. Sedimentation follows immediately after preliminary treatment and it uses the same basic principles. The chief aim is to remove settle able solids in wastewater.

To determine whether a sedimentation tank is necessary, a test carried out using Imhoff cones. In the case of domestic wastewater, there is removal of about 60-70 per cent of the settle able solids. BOD removal can be between 30 and 40 per cent. Generally, primary sedimentation is economical and efficient for the removal of solids.

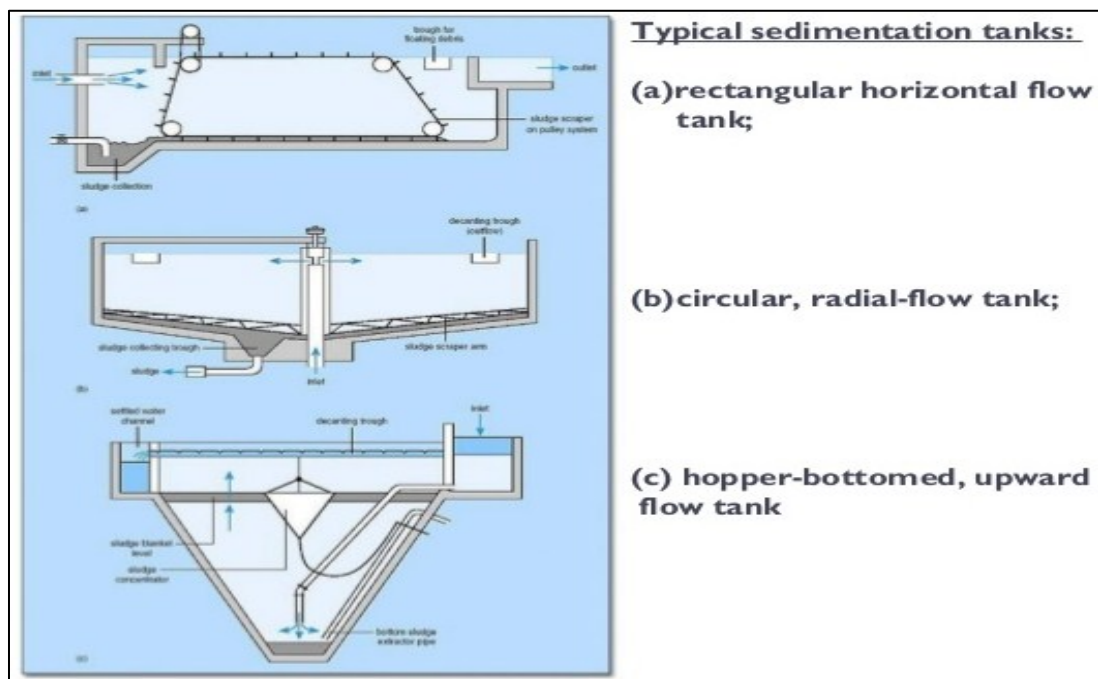


Figure (2.5): Typical Sedimentation Tanks and Clarifiers used in WWTP [4].

2.4 Unit Operation of Secondary Treatment (Biological Treatment).

The major purpose of secondary treatment is to remove the soluble BOD that escapes primary treatment and to provide further removal of suspended solids. The basic ingredients needed for convention aerobic secondary biological treatment are the availability of many microorganisms, good contact between these organisms and the organic material, the availability of oxygen, and the maintaining other environmental condition [2].

2.4.1 Aeration Tank.

The use of biological processes to provide further treatment referred to as secondary treatment. The wastewater, having had most of the solids removed in primary treatment, flows on to an aeration basin. Air added to the aeration basins to create an environment for beneficial/helpful microorganisms to grow and continue treating the remaining pollutants in the wastewater. These microorganisms "bugs" continue to consume/treat any remaining dissolved organic material in the water. These bugs get their name from the fact that they are so small you need a microscope to see them. The process of microbes eating waste products actually begins in the sewer pipes along the way to the plant. However, aeration basins provide the perfect environment for maximum consumption of the organic wastes. The process of using microbes to consume wastes in this oxygen- rich environment called activated sludge. Activated sludge looks like dark mud. It is rich with active ("live") microbes: bacteria and protozoan. The bacteria and protozoans require oxygen to live and thrive, just like other living organisms. The aeration basin gives them extra oxygen so they will grow and consume the waste. The food chain is a continual cycle. In the aeration basins, the microbes, the "work force" of a wastewater plant, digest and break down organic material and

then die out when they reach the end of their life cycle. Since new wastewater, carrying new organic material is added all the time, the cycle continues and new Microbes are born.



Figure (2.6): Diffused Aeration [5].

A variety of treatment methods have used in the past to meet these basic needs. The most common treatment methods are:

- 1) Trickling filter.
- 2) Activated sludge.
- 3) Rotating Biological Contactor or RBC's.
- 4) Lagoons.

1) Tricking Filters.

Tricking filters have been a popular biological treatment process. The most widely used design for many years was simply a bed of stones from 1 to 3 m deep through which the wastewater passed. The wastewater is typically distribution over the surface of the rocks by a rotating arm figure (8).

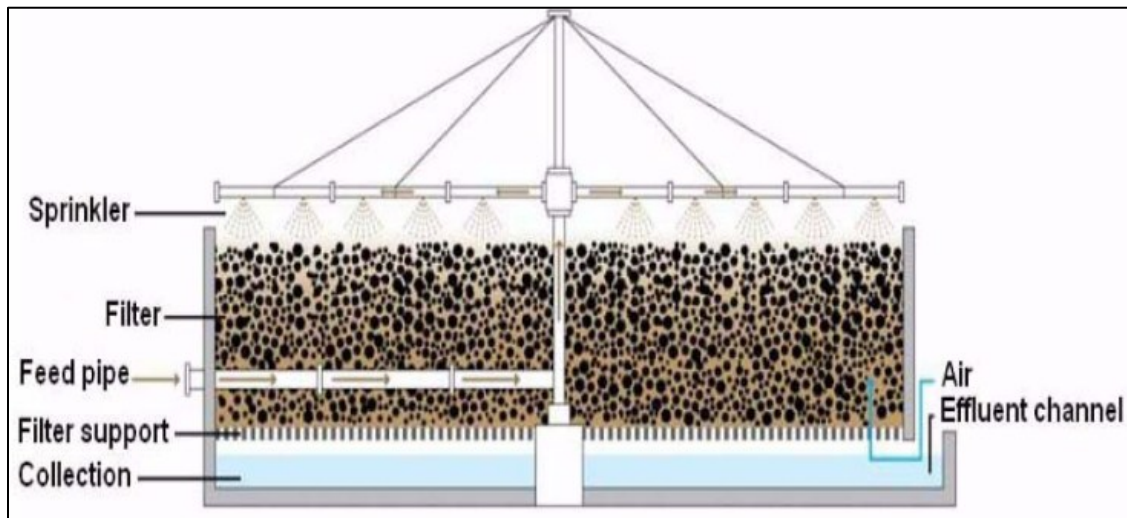


Figure (2.7): Trickling Filter Including its Components [5].

Trickling filters are not primarily a filtering or straining process as the name implies. The rocks in a rock filter are 25 to 100 mm in diameter, and hence, have openings too large to strain out solids. They are means of providing large amounts of surface area where the microorganisms cling and grow in slime on the rocks as they feed on the organic matter.

The excess growth of microorganisms wash from the rock media, and can cause undesirable high levels of suspended solids in the plant effluent if not removed. Thus, the flow from the filter passed through a sedimentation basin to allow these solids to settle out. This sedimentation basin referred to as a secondary clarifier or final clarifier to differentiate it from the sedimentation basin used for primary settling. To overcome these laminations, other materials have become popular for filling the trickling filter. These materials include modules of corrugated plastic sheets and plastic rings. These media offer larger surface areas for slime growths (typically 90 square meters of surface area per cubic meter for 75mm rocks) and greatly increase void ratio for increased airflow. The materials are also much lighter than rock (by a factor of about 30), so that the trickling filters, can be much

taller without facing structural problems. While rock in filters is usually not more than 3m deep, synthetic depths may reach 12 m, thus reducing the overall space requirements for the trickling-filter portion of the applied hydraulic plant.



Figure (2.8): Trickling Filter [7].

When using a trickling filter, primary sedimentation is necessary as a pre-treatment to remove solids. A reasonably uniform application of wastewater (intermittent) over the media is necessary for effective treatment. Correct media size and shape are necessary. If media are too small, the filter may be blocked easily. The normal size varies from 25 mm to 100 mm. The filter should have a good under-drain for complete effluent removal. This ensures proper ventilation and provides support for the media. There should be ecological balance as well as balance of microorganisms. The design should facilitate proper hydraulic or organic loading.

There should be no difficulty in starting the operation of a well-constructed trickling filter other than the time lag required to build up (develop) an active film of micro-organisms. This will establish itself in a few weeks. Industrial waste is not always capable of forming the necessary biological growth, and it is common practice to start industrial waste filtering with sewage or a sewage product such as septic tank effluent. Once a trickling filter is in operation, waste flow to the filter

should not discontinued because even a few hours of inactivity will reduce the biological efficiency, and biomass may decrease.

Clogging is a condition of local flooding within the filter bed .It occurs when liquid is unable to pass through the bed. At the top surface, it appears as a pond between dosing. Within the filter, clogging is less obvious. In other locations, it interferes with filter operation since passage of air (oxygen) through the clogged region is retarded and anaerobic conditions may develop. Effluent qualities under such conditions may be considerably impaired.

Clogging may cause by too fine a stone size (media) in the bed, or it may be from too wide a range of stone sizes since this result in a closer parking, which increases free area of liquid movement. Clogging may occur because of the nature of the waste For example, excessive quantities of suspended solids in the wastewater fed to the filter causes overloading and building up of deposits on or within the bed. In case, there is clogging, rest the unit for a few days to dry the flock and slough off. When wastewater passed through again, it washes already sloughed materials.

Small grey flies are very prevalent around tricking filters; they are referred as “filter flies “. They are a nuisance because of their large numbers. Chlorinating the waste entering the filter may keep the flies under control. Also overdosing with chlorine may kill some other microorganisms. Since the whole action of the trickling filter based on biological growth, any toxic materials that affect health and activity of microorganisms will impair the efficiency of the filter temporarily or permanently. In municipal sewage treatment, strict limitations on discharge of toxic waste into sewers needed in order to prevent disturbances of the treatment process. A reasonable degree of toxic materials can developed in the filter biomass

by gradual build-up of toxic concentration, but shock loads or sudden surges of concentrated poison are often disastrous to filter efficiency [8].

2.4.2 Activated Sludge:

The activated sludge process is a biological wastewater treatment technique in which a mixture of wastewater and biological sludge (microorganisms) is agitated and aerated. The biological solids are subsequently separated from the treated wastewater and returned to the aeration process derives its name from the biological mass formed when air is continuously injected into the wastewater. Under such conditions, microorganisms are mixed thoroughly with the organics under conditions that simulation their growth through use of the organic as food. As the microorganisms grow and are mixed by the agitation of the air, the individual organisms clump together, and (flocculate) to form an active mass of microbes (biological floc) called activated sludge.

In practice, wastewater flows continuously into an aeration tank where air injected to mix the activated sludge with the wastewater and the supply the oxygen needed for the organisms to break down the organics. The mixture of activated sludge and wastewater in the aeration tank called mixed liquor. The mixed liquor flows from the aeration tank to secondary clarifier activated sludge settled out. Most of the settled sludge returned to the aeration tank and hence called return sludge to maintain a high population of microbes to permit rapid breakdown of the organics. Because more activated sludge produced than is desirable in the process, some of the return sludge diverted or wasted to the sludge handling system for treatment and disposal.

In conventional activated sludge system, the wastewater typically aerated for six to eight hour in long, rectangular aeration basin [6].

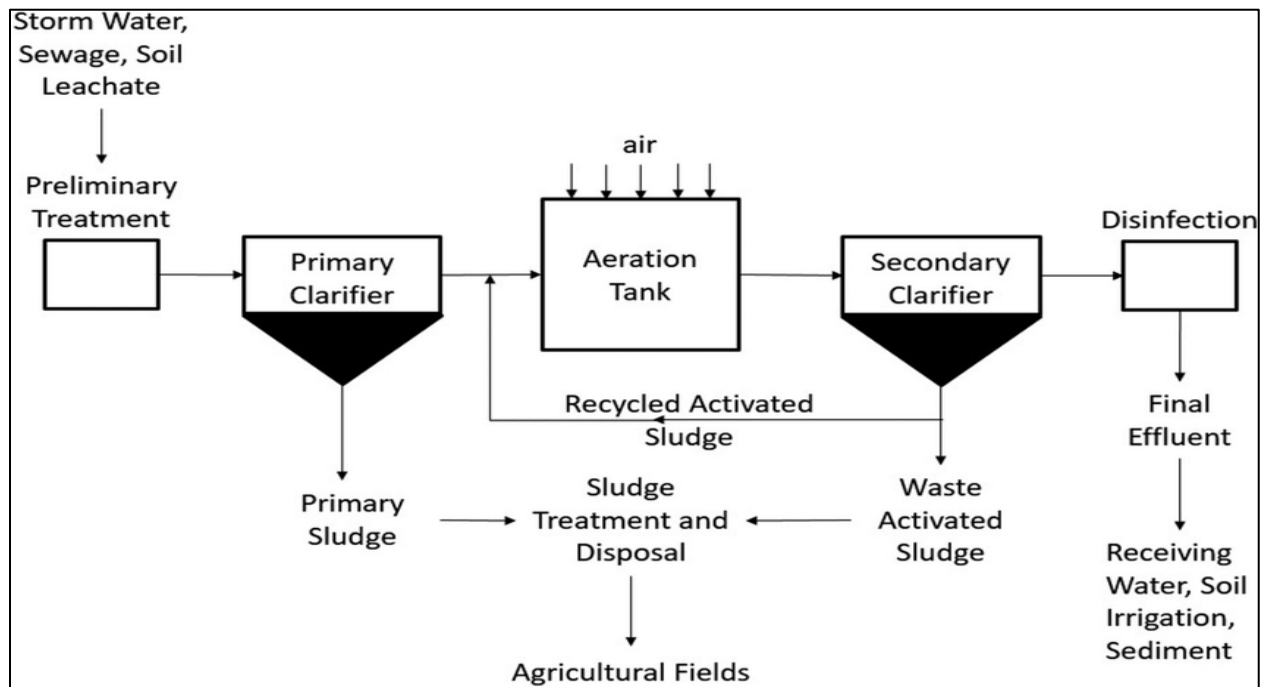


Figure (2.9-a): Schematic Diagram Showing Activated Sludge Treatment System [7].



Figure (2.9-b): Aeration Tanks in an Activated Sludge Treatment [9].

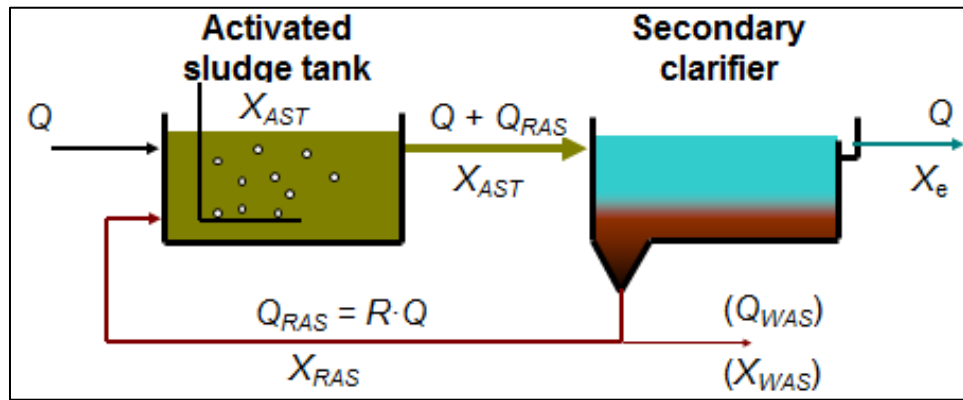


Figure (2.9-c): Completely Mix Activated Sludge System with Recycle [10].

2.4.3 Rotating Biological Contactor or RBC's.

RBC is a biological treatment process used in the treatment of wastewater following primary treatment. The RBC process involves allowing the wastewater to be exposed to a biological medium in order to remove pollutants in the wastewater before discharge of the treated wastewater to the environment, usually a body of water (river, lake or ocean). A rotating biological contactor consists of a series of closely spaced, parallel discs mounted on a rotating shaft, which is supported just above the surface of the wastewater. Microorganisms grow on the surface of the discs where biological degradation of the wastewater pollutants takes place [4].

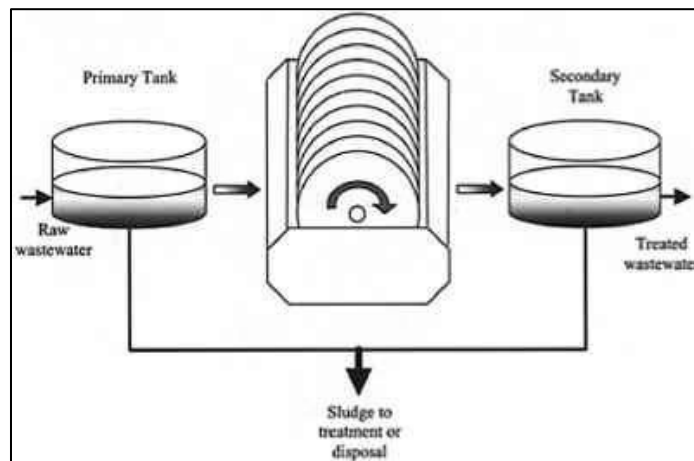


Figure (2.10-a): A Single Rotating Biological Contactor or RBC [10].

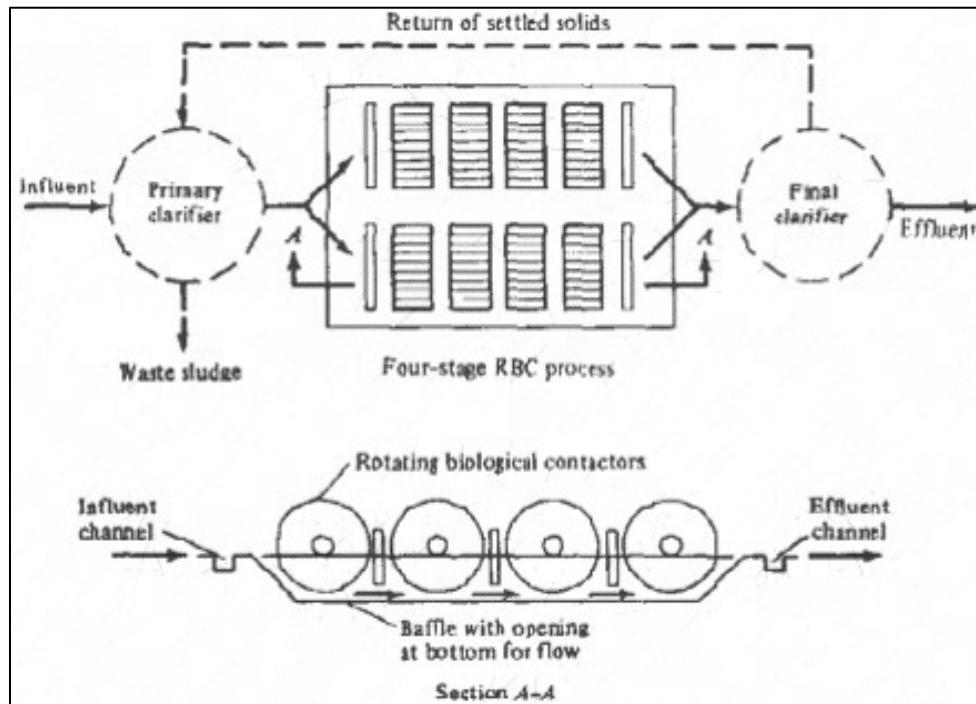


Figure (2.10-b): A Group of Rotating Biological Contactors [11].

2.4.4 Wastewater Stabilization Lagoons.

It is no wonder that one of the most popular methods for wastewater treatment around the world is also one of the simplest and least expensive. Lagoon systems use natural and energy-efficient processes to provide low-cost wastewater treatment. They are one of the most cost-effective wastewater treatment options for many homes and communities. In the U.S., most of wastewater treatment lagoons found in small and rural communities. Lagoons are especially well suited to small communities because they can cost less to construct, operate, and maintain than other systems. They also require more land than other wastewater treatment methods, and land is more likely to be available and inexpensive in rural areas. Lagoons can also be designed to serve individual households. They sometimes are a good option for homes on large lots in areas where other onsite systems or sewers are too costly or otherwise impractical. Lagoons also work well for many seasonal rental properties and recreational areas, because they are able to handle intermittent

periods of both light and heavy use. What are lagoon systems? There are several different types and names for lagoons and many possible system designs. Lagoon systems include one or more pond-like bodies of water or basins designed to receive, hold, and treat wastewater for a predetermined period. Lagoons are constructed and lined with material, such as clay or an artificial liner that will prevent leaks to the groundwater below. While in the lagoon, wastewater receives treatment through a combination of physical, biological, and chemical processes. Much of the treatment occurs naturally, but some systems designed to use aeration devices that increase the amount of oxygen in the wastewater also. Aeration makes treatment more efficient, so that less land area is necessary, and aerators can be used to upgrade some existing systems to treat more wastewater. Every lagoon system must be individually designed to fit its specific site and use. Designs based on such factors as the type of soil, the amount of land area available, the climate, and the amount of sunlight and wind in an area. Other important design considerations for lagoon systems include the amount and type of wastewater to be treated and the level of treatment required by state and local regulations. Depending on local standards and the final method of disposal chosen, wastewater leaving lagoon systems often requires additional treatment, or “polishing,” to remove disease-causing organisms or nutrients from the wastewater before it can be returned to the environment.

This issue of pipeline includes a brief overview of some of the different types of wastewater treatment lagoons, how they work, their operation, maintenance, and some of their advantages and disadvantages. Only a few of the many possible classifications and designs for lagoon systems mentioned here.

Lagoons are not all the same. Some employ different biological, chemical, and physical processes to treat the wastewater, while others may play a different role in

overall treatment. Some lagoon designs provide adequate treatment for certain methods of discharge, while others should use in combination with other lagoons or with additional treatment. Complicating matters further, there can be several different names for the same type of lagoon. For example, the terms lagoon and pond often used interchangeably, and names, such as polishing, stabilization, and maturation, can refer to a lagoon's particular role in treatment. This can be very confusing for community leaders and homeowners trying to evaluate lagoon systems. The following is a brief overview of some of the more common types of lagoons and some of the terms used for them.

2.4.5 Anaerobic Lagoons.

The word anaerobic means without oxygen, which describes the conditions inside this type of lagoons. Anaerobic lagoons most often used to treat animal wastes from dairies and pig farms, commercial or industrial wastes, or as the first treatment step in systems using two or more lagoons in a series. Typically, anaerobic lagoons are designed to hold and treat wastewater from 20 to 50 days.* they are relatively deep (usually 8 to 15 feet) and work much like septic tanks. Inside an anaerobic lagoon, solids in the wastewater separate and settle into layers. The top layer consists of grease, scum, and other floating materials. This layer keeps oxygen out, allowing bacteria and other organisms that thrive in anaerobic conditions to work to treat the wastewater. As with septic tanks and most other lagoon designs, the layer of sludge that settles at the bottom of an anaerobic lagoon eventually accumulates and must be remove periodically. Also similar to a septic tank, the wastewater that leaves an anaerobic lagoon always requires further treatment. Odor can be a problem with anaerobic lagoons. However, in many cases odor can managed through a variety of methods, such as adding sodium nitrate, recirculating pond effluent, and through regular maintenance.

2.4.6 Aerobic Lagoons.

Dissolved oxygen is present throughout much of the depth of aerobic lagoons. They tend to be much shallower than other lagoons, so sunlight and oxygen from air and wind can better penetrate the wastewater. In general, they are better suited for warm, sunny climates, where they are less likely to freeze. Wastewater usually must remain in aerobic lagoons from 3 to 50 days to receive adequate treatment.* Wastewater treatment takes place naturally in aerobic lagoons with the aid of aerobic bacteria and algae. Because they are so shallow, their bottoms need to be either paved or lined with materials that will prevent weeds from growing in them. Sometimes, the wastewater in aerobic lagoons needs to be mixed to allow sunlight to reach all of the algae and to keep it from forming a layer that blocks out the air and sun completely.

2.4.7 Aerated Lagoons.

Aerated lagoons are very common in small communities. These systems use aerators to mix the contents of the pond and add oxygen to the wastewater. They are sometimes referred to as partial-mix or complete-mix lagoons depending on the extent of aeration. Partial-mix aerated lagoons are sometimes facultative lagoons that have been adapted and upgraded to receive more wastewater. With the exception of wind-driven designs, most aerators require energy to operate. However, energy costs are usually considerably less than those for other mechanical community treatment systems.

Aeration makes treatment more efficient, which offsets energy costs in some cases. Aerated lagoons require less land area and shorter detention times for wastewater than other lagoons.

2.4.8 Facultative Lagoons.

Both aerobic and anaerobic conditions exist in facultative lagoons, which are also called stabilization ponds, oxidation ponds, photosynthetic ponds, and aerobic-anaerobic ponds. They are the most common type of wastewater treatment lagoon used by small communities and individual households. Facultative lagoons can be adapted for use in most climates, require no machinery, and treat wastewater naturally, using both aerobic and anaerobic processes. Because they are used so often by small communities, Lagoons in general, rely on the interaction of sunlight, algae, microorganisms, and oxygen (sometimes aerated) (Pipeline 1997). A process in which plants utilize sun light and chlorophyll to convert carbon dioxide and inorganic substances to oxygen and additional plant material as shown on figure(2.11) (through) [13].

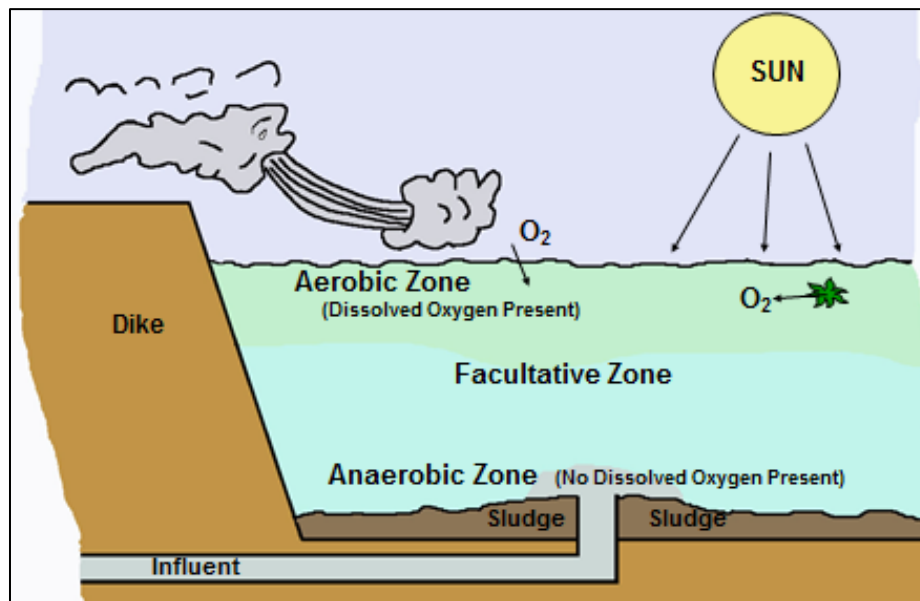


Figure (2.11): Zonal Relationship in Lagoons [13].

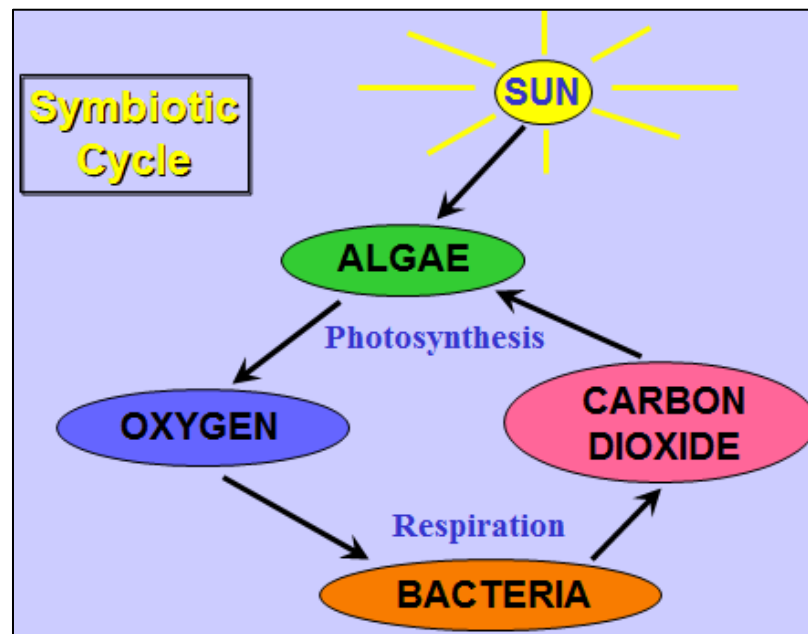


Figure (2.12): Treatment Symbiotic Cycle in Lagoon System [13].

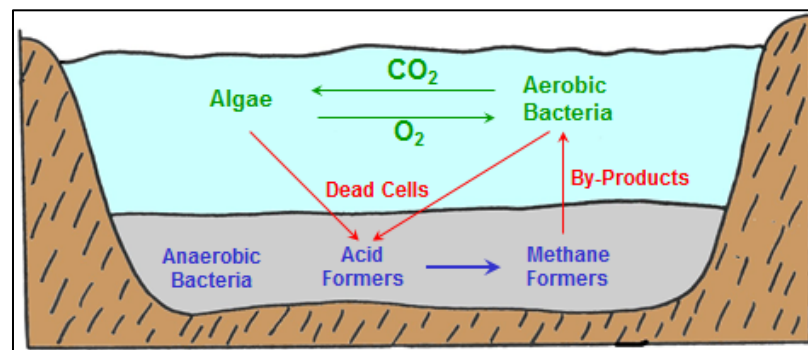


Figure (2.13): Activities in Facultative Lagoons [13].

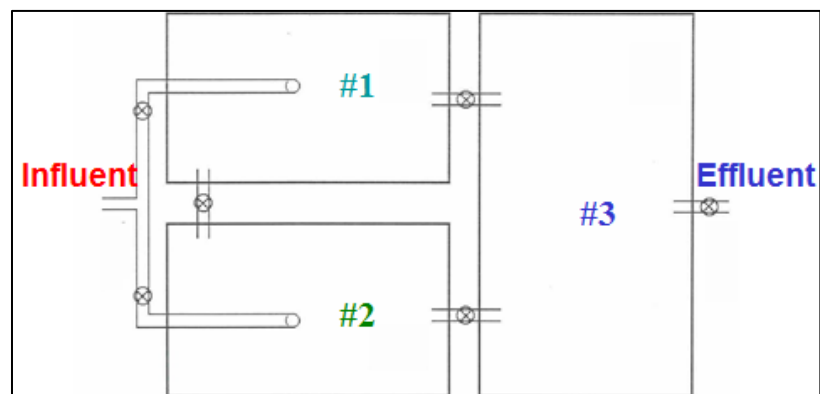


Figure (2.14): Typical Flow Diagram for a Lagoon System [13].

2.4.9 Advantages of Lagoon System.

- Lagoon systems can be cost-effective to design and construct in areas where land is inexpensive.
- They use less energy than most of wastewater treatment methods.
- They are simple to operate, maintain, and generally require only part-time staff.
- They can handle intermittent use and shock loadings better than many systems, making them a good option for campgrounds, resorts, and other seasonal properties.
- They are very effective at removing disease-causing organisms (pathogens) from wastewater.
- The effluent from lagoon systems can be suitable for irrigation (where appropriate), because of its high nutrient and low pathogen content.

2.4.10 Disadvantages of Lagoon System.

- Lagoon systems require more land than other treatment methods.
- They are less efficient in cold climates and may require additional land or longer detention times in these areas.
- Odor can become a nuisance during algae blooms, spring thaw in cold climates or with anaerobic lagoons and lagoons that inadequately maintained.
- Unless they properly maintained, lagoons can provide a breeding area for mosquitoes and other insects.
- They are not very effective at removing heavy metals from wastewater.
- Effluent from some types of lagoons contains algae and often requires additional treatment or “polishing” to meet local discharge standards.
- Possible groundwater contamination.

- Not perfect in high loading situations, it require large land usage.

2.4.11 Results of wastewater treatment by lagoon system

Well-designed wastewater treatment system using lagoon system will accomplish the following results:

- Public health is protected because of pathogens exist in wastewater is removed.
- Environment protected because the characteristics of wastewater are changed, and the end products are stabilized.
- Process itself is not offensive.

2.4.12 Anaerobic digestion

Scientific Overview Anaerobic digestion is a natural biological process. The initials “AD” may refer to the process of anaerobic digestion, or the built systems of anaerobic digesters. While there are many kinds of digesters, the biology same for all. Anaerobic digesters built systems that deliberately harness the natural process. AD systems can minimize odors and vector attraction, reduce pathogens, produce gas, produce liquid and solid digestate, and reduce waste volumes. Anaerobically digesting organic carbon involves naturally occurring bacteria. Digestion takes place when organic materials decompose in an oxygen-free environment.

Some digester systems differentiate between “wet” and “dry” digesters, or low-solid and high-solid systems, and sometimes the process called fermentation. The different languages used to describe the same processes reflect the varied historical uses and development of AD. Figure (16), shows the various components of the anaerobic digestion system.

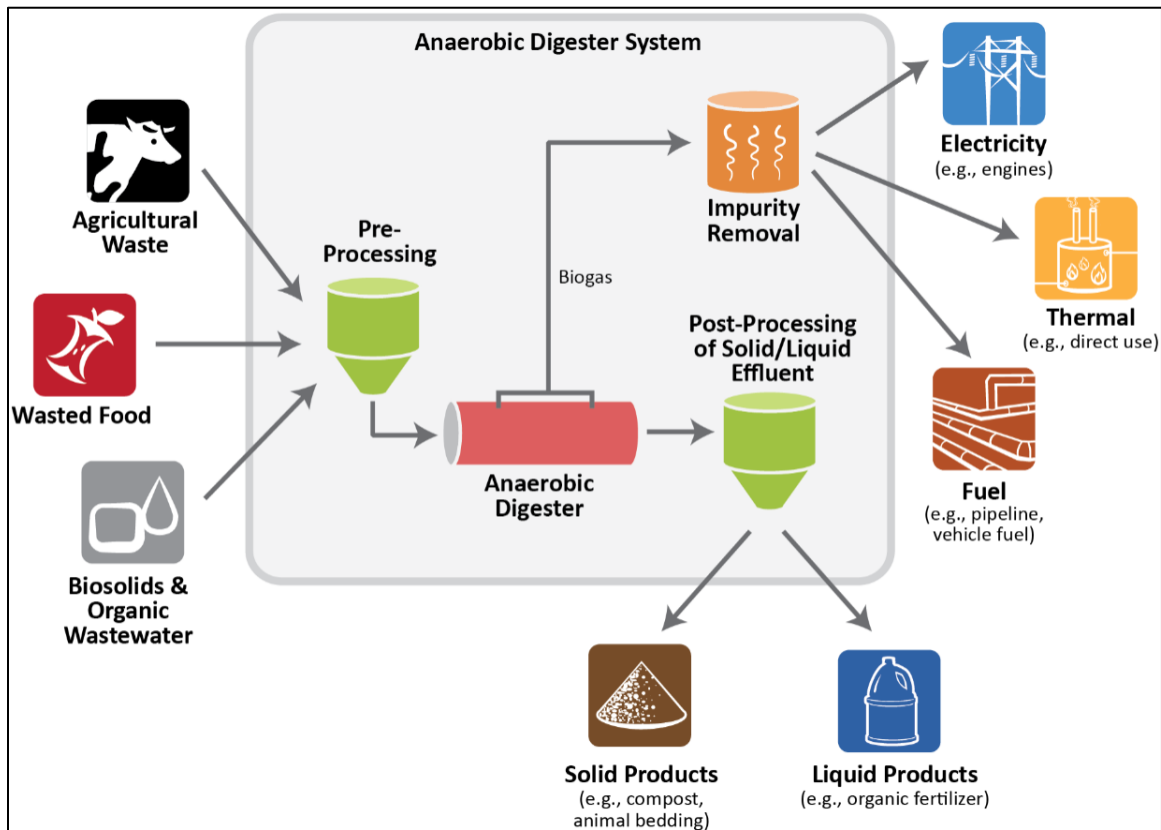


Figure (2.15): Anaerobic System Components [14].

During digestion, various microbes use the organic matter such as animal manure, sewage sludge, wasted food and other organics in the absence of oxygen. The process can be controlled and enhanced through chemistry and engineering. The chemical reactions that occur in stages during anaerobic digestion are hydrolysis, fermentation, also called acido-genesis (the formation of soluble organic compounds and short-chain organic acids), and methano-genesis (the bacterial conversion of organic acids into methane and carbon dioxide) (Metcalf & Eddy, 2003). In the methano-genesis step, acetic acid, carbon dioxide, and hydrogen converted to biogas by methanogens. Biogas consists mainly of methane and carbon dioxide and can be used as a renewable energy fuel in a variety of applications [14].

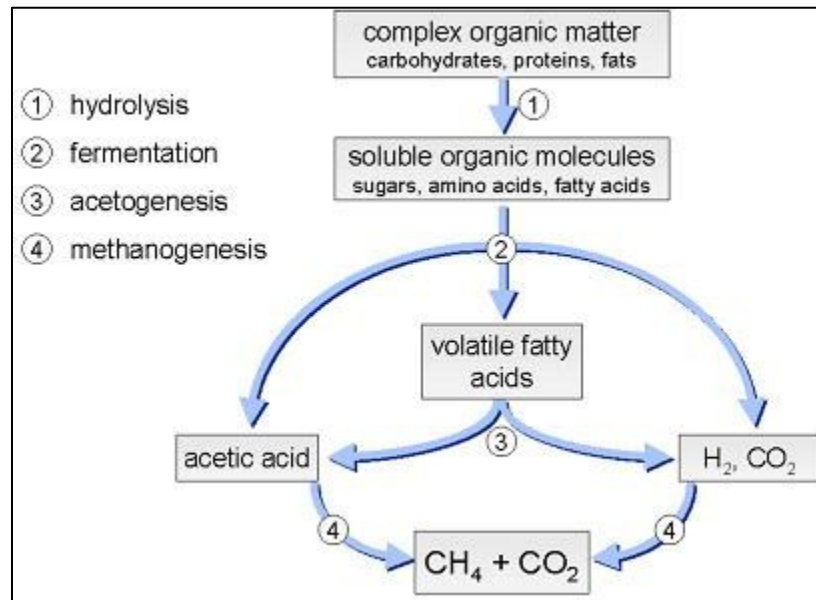


Figure (2.16): The Chemical Reactions That Occur During Anaerobic Digestion [14].

2.5 Applications of anaerobic digester systems (AD)

Generalizations about anaerobic digester systems often overlook variations. There are many sizes, styles, and applications of digesters. AD systems can be house-sized or town-sized. They can be used primarily for waste processing or energy generation.

Anaerobic digester systems can be designed to optimize mixing, volume reduction, biogas production, pathogen destruction, vector attraction reduction, and odor control. Systems can be designed as batch or continuous flow systems, within a sealed vessel or holding tank, or with a series of vessels.

Anaerobic digestion processes come in different configurations. Low rate anaerobic digesters are usually used for small systems (under 1 million gallons per day), usually contain no auxiliary mixing, and are operated at long sludge retention times (SRTs) in the 30-60 day range. High rate systems are more commonly used and characterized by supplemental heating, auxiliary mixing, uniform feeding rates,

and sludge thickening before digestion (WEF, 1998). They are designed for mesophilic temperatures (86-100° F), the most common configuration in North America, at SRTs in the 12-25 day range; or thermophilic temperatures (122-140°F) at SRTs in the 10-12 day range.

Two-stage anaerobic digester systems include a first stage (mesophilic or thermophilic), where most of the gas is produced, and a second stage used for solid-liquid separation or as a holding tank before dewatering. Temperature-Phased anaerobic digestion configurations combine in one system both mesophilic and thermophilic digestion stages connected in series and can offer significant advantages including significantly improved volatile solids reduction and biogas production. Two-phase AD systems are also available with the first stage being an acid phase reactor and the second phase being a methano-genic reactor. Three and multi-stage configurations are available. Each AD alternative has advantages and disadvantages (Kalogo and Monteith, 2008).

Digesters can handle a variety of feedstock. Some digesters designed for one feedstock. However, may be adapted to other feedstock or combination of them. Co-digestion of sewage sludge with other feedstocks (e.g. fats, oils & grease (FOG), wasted food, cheese or wine wastes, manure) can increase biogas production. Co-digestion can significantly increase biogas production and possibly volatile solids reduction depending on the type of organic feedstock added and other factors (Parry, 2014).

Economic feasibility of co-digestion is strongly dependent upon waste characteristics, regional energy costs, and bio-solids residual management costs. Most waste streams (perhaps with the exception of FOG) require a tipping fee paid to the digester owner to achieve economic feasibility. Facilities considering co-digestion should consider utilizing existing process capacity prior to exploring

construction of additional capacity for co-digestion (Parry, 2014). Feedstock converted to biogas and digested material, which reduces their volume. The volume reduction and gas production is dependent on the specific feedstock and process.

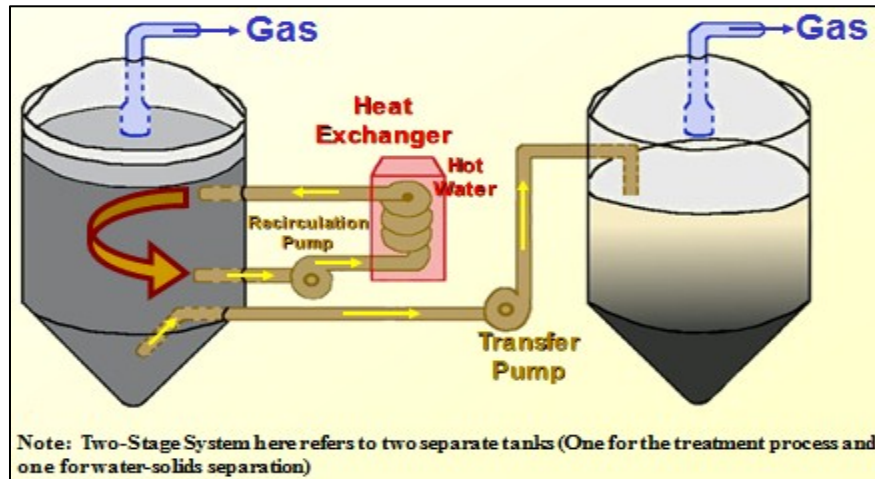


Figure (2.17): Typical Two Stage Anaerobic Digester System [14].

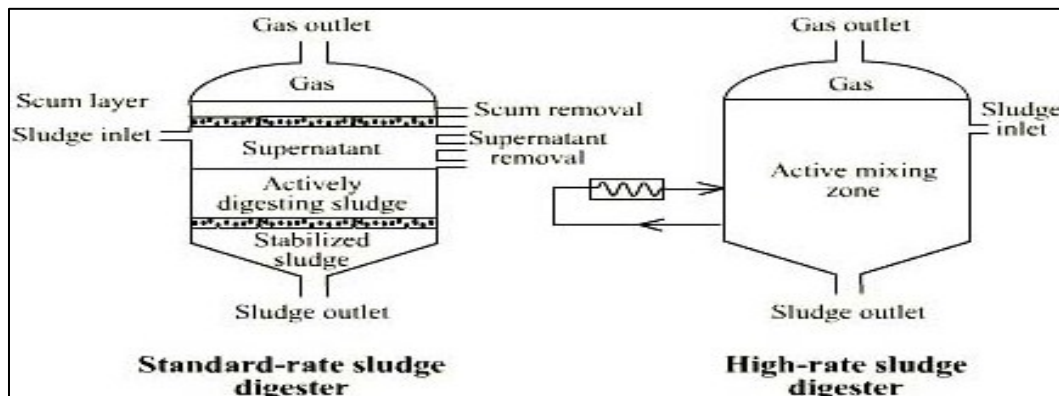


Figure (2.18): Standard Rate and High Rate Sludge Digester [14].

2.6 Tertiary Treatment.

Simply additional treatment beyond secondary! tertiary treatment can remove more than 99 percent of all the impurities from sewage, producing an effluent of almost drinking- water quality. The related technology can be very expensive, requiring a high level of technical “know-how” and well treatment plant operators, a steady

energy supply, and chemicals and specific equipment, which may not be readily available. An example of a typical tertiary treatment process is the modification of a conventional secondary treatment plant to remove additional phosphorus and nitrogen. Disinfection, typically with chlorine, can be the final step before discharge of the effluent. However, some environmental authorities are concerned that chlorine residuals in the effluent can be a problem in their own right, and have moved away from this process. Disinfection frequently built into treatment plant design, but not effectively practiced, because of the high cost of chlorine, or the reduced effectiveness of ultraviolet radiation where the water is not sufficiently clear or free of particles.

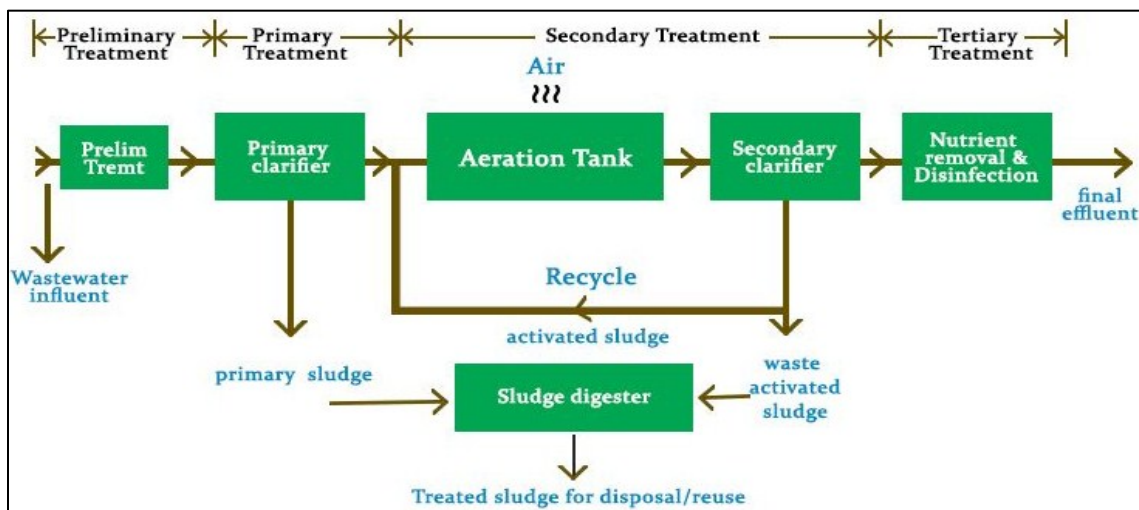


Figure (2.19-a): Stages of Domestic Wastewater Treatment Plant [5].

Secondary treatment removes 85 to 95 percent of BOD and TSS and minor portions of nitrogen, phosphorus, and heavy metals. Tertiary treatment is the next wastewater treatment process after secondary treatment. This treatment sometimes called as the final or advanced treatment, which consists of removing the organic load left after secondary treatment for removal of nutrients from sewage. In addition, particularly to kill the pathogenic bacteria. The effluents from secondary

sewage treatment plants contain both nitrogen (N) and phosphorus (P). N and P are ingredients in all fertilizers.

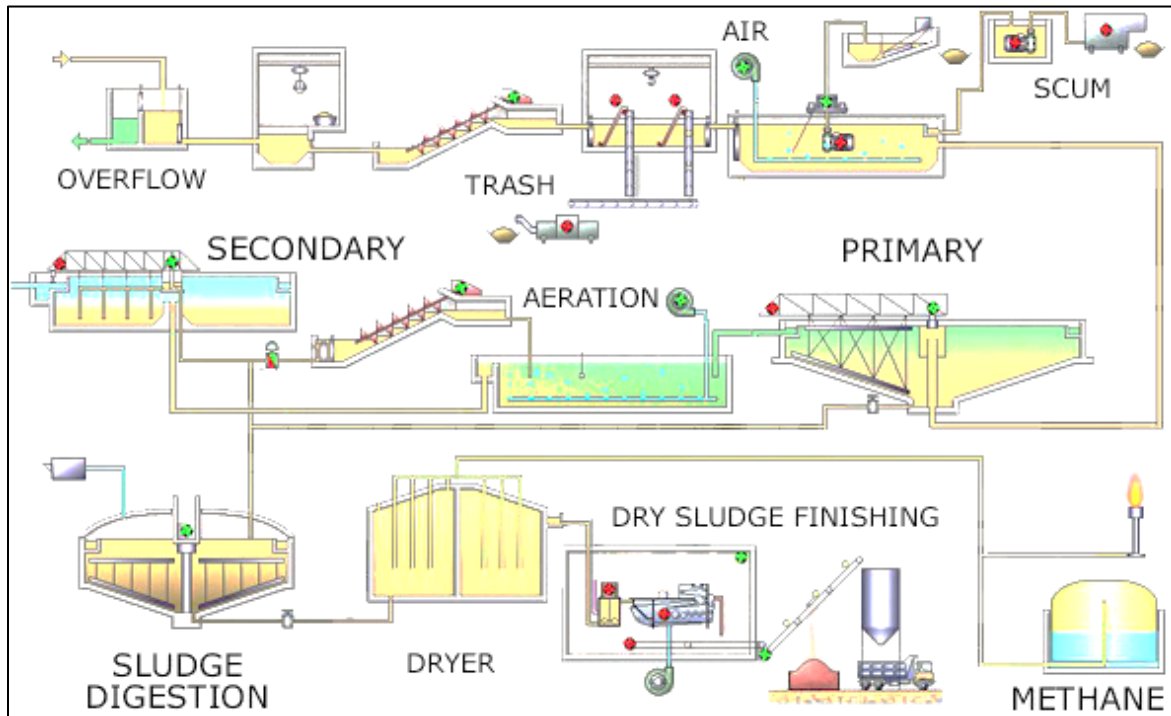


Figure (2.19-b): Schematic Typical Domestic Wastewater Treatment Plant [4].

When excess amounts of N and P discharged, plant growth in the receiving waters may be accelerated which results in eutrophication in the water body receiving such waste. Algae growth may be stimulated causing blooms, which are toxic to fish life as well as aesthetically displeasing. Secondary treated effluent also contains suspended, dissolved, and colloidal constituents, which may require removal for stipulated reuse or disposal of the treated effluent. 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 such as sea, river, lake, ground, etc., or to raise the treated water quality to such a level to make it suitable for intended reuse. This step removes different types of pollutants such as organic matter, SS, nutrients, pathogens, and heavy metals that secondary treatment is not able to

remove. Wastewater effluent becomes even cleaner in this treatment process using stronger and more advanced treatment systems. It includes sedimentation, coagulations, membrane processes, filtration, ion exchange, activated carbon adsorption, electro dialysis, nitrification and denitrification, etc. (Figure 2.19-b). Tertiary treatment is costly as compared to primary and secondary treatment methods. Need of Tertiary treatment Tertiary treatment may provide to the secondary effluent for one or more of the following contaminant further.

- To remove total suspended solids and organic matter those are present in effluents after secondary treatment.
- To remove specific organic and inorganic constituents from industrial effluent to make it suitable for reuse. 3 | Page NPTEL IIT Kharagpur Web Courses
- To make treated wastewater suitable for land application purpose or directly discharge it into the water bodies like rivers, lakes, etc.
- To remove residual nutrients beyond what can be accomplished by earlier treatment methods.
- To remove pathogens from the secondary treated effluents.
- To reduce total dissolved solids (TDS) from the secondary treated effluent to meet reuse quality standards.

One or more of the unit operation/ process mentioned in Figure (2.19-b) will be used for achieving this tertiary treatment. Figure (2.19-b) Process involved in tertiary wastewater treatment in advanced wastewater treatment, treatment options or methods are dependent upon the characteristics of effluent to be obtained after secondary treatment to satisfy further use or disposal of treated wastewater. The principal nitrogen compounds in domestic sewage are proteins, amines, amino acids, and urea. Ammonia nitrogen in sewage results from the bacterial decomposition of these organic constituents. The nitrogen compounds result from

the biological decomposition of proteins and from urea discharged in body waste. This nitrogen is in complex organic molecules and referred simply as organic nitrogen. Organic nitrogen may be biologically converted to free ammonia (NH_3) or to the ammonium ion (NH_4^+) by one of several different metabolic pathways. These two exist in equilibrium as $\text{NH}_4 + \text{NH}_3 + \text{H}^+$. Ammonia nitrogen is the most reduced nitrogen compound found in wastewater, which can be biologically oxidized to nitrate if molecular oxygen is present (under aerobic condition). In wastewater, the predominant forms of nitrogen are organic nitrogen and ammonia. The nitrification may take place in biological treatment units provided the treatment periods are long enough. Generally, for the HRT used in secondary treatment conversion of organic nitrogen to ammonia is significant and nitrification may not be significant. Because of oxygen demand exerted by ammonia (about 4.6 mg of O_2 per mg of NH_4^+ -N oxidized) and due to other environmental factors, removal of ammonia may be required. The most common processes for removal of ammonia from wastewater are i) Air stripping, ii) Biological nitrification and denitrification [9].

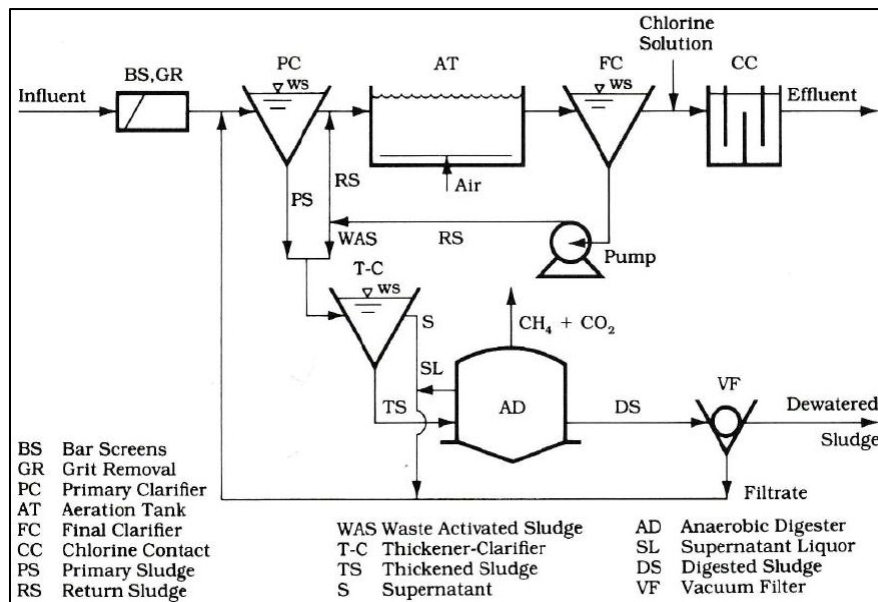
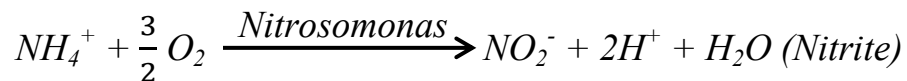


Figure (2.19-c): Process Flow Diagram of a Typical Domestic WWTP [5].

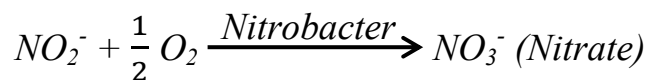
2.6 Biological Nitrification and De-nitrification.

Bacteria remove ammonia nitrogen from wastewater by two-step biological processes: nitrification followed by de-nitrification to convert it finally to gaseous nitrogen. In this gaseous form, N_2 is inert and does not react with the wastewater itself or with other constituents present in wastewater. Since, treated wastewater likely to be saturated with molecular nitrogen; the produced N_2 is simply released to the atmosphere. These two steps involved require different environmental conditions and hence generally they carried out in separate reactors. Nitrification has important role in nitrogen removal from wastewater during treatment. The biological conversion of ammonium to nitrate nitrogen called Nitrification. It is autotrophic process i.e. energy for bacterial growth is derived by oxidation of nitrogen compounds such as ammonia. In this process, the cell yield per unit substrate removal is smaller than heterotrophs. Nitrification is a two-step process. In first step, bacteria known as Nitro-somonas can convert ammonia and ammonium to nitrite. These bacteria known as nitrifiers are strictly aerobes. This process limited by the relatively slow growth rate of Nitro-somonas. Next, bacteria called Nitro-bacter finish the conversion of nitrite to nitrate.

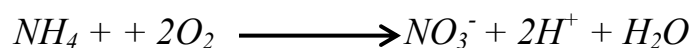
First step:



Second step:



Overall reaction:



Nitro-somonas and Nitro-bacter use the energy derived from the reactions for cell growth and maintenance. Some of ammonium ions assimilated into cell tissues.

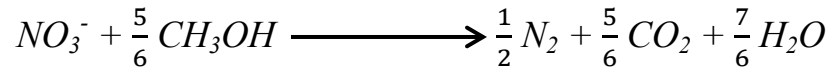
Neglecting this ammonium ion used in cell synthesis the O_2 required to oxidize ammonia to nitrate is 4.57 mg O_2 /mg ammonium nitrogen. If the ammonium used in cell, O_2 required considered it is 4.3 mg O_2 /mg ammonium nitrogen and about 7.14 mg of alkalinity needed to neutralize the H^+ produced [17].

Nitrification may be use to prevent oxygen depletion from nitrogenous demand in the receiving water. Nitrification requires a long retention time, a low food to microorganism ratio (F/M), a high mean cell residence time (MCRT), and adequate alkalinity. Wastewater temperature and pH affects the rate of nitrification. Under favorable conditions, carbon oxidation and nitrification may occur in a single reactor called single stage under favorable conditions. In separate stage, carbon-oxidation and nitrification occur in different reactors. It can be accomplished in both suspended and attached growth process such as trickling filter, ASP, rotating disc biological contactor (RBC), SBR, etc. In addition, nitrifying organisms are present in almost all-aerobic process sludge but they are less in number. In nitrification, when ratio of BOD_5 to TKN is greater than 5, the process called as combined carbon oxidation and nitrification, whereas, when ratio of BOD_5 to TKN is less than 3, the process called as separate stage carbon-oxidation and nitrification. For single stage carbon oxidation and nitrification, plug flow, completely mixed and extended aeration ASP can used. Oxidation ditch is also one of option for this process. The attached growth processes like TF and RBC are commonly used. Nitrification can achieved by reducing applied loading rate, increasing the mean cell residence time (θ_c) and air supply. Nitrifying bacteria are sensitive organisms. A variety of organic and inorganic agents can inhibit the growth and action of these organisms. High concentration of ammonia and nitrous acid can be inhibitory. The effect of pH is also significant with optimal range of 7.5 to 8.6. The system acclimatize to lower pH can also work successfully. The

temperature also has considerable impact on growth of the nitrifying bacteria. Dissolved oxygen concentration above 1 mg/L is essential for nitrification. Below this DO, oxygen becomes the limiting nutrients and nitrification slows down or ceases. De-nitrification In some applications, such as discharge of effluent into enclosed water bodies or recycle to water supplies, nitrification may not be sufficient. When nitrogen removal is required, one of the available methods is to follow biological nitrification with de-nitrification. De-nitrification is Nitrates removed by two mechanisms: (1) conversion of NO_3 to N_2 gas by bacterial metabolism; (2) conversion of NO_3 to nitrogen contained in cell mass, which may be remove by settling.

De-nitrification occurs when oxygen levels depleted and nitrate becomes the primary electron acceptor source for microorganisms. Nitrate, NO_3 - Nitrite, NO_2 - Nitric oxide, NO Nitrous oxide, N_2O Nitrogen, N_2 Denitrifying bacteria are facultative organisms, they can use either dissolved oxygen or nitrate as an oxygen source for metabolism and oxidation of organic matter. This carried out by hetetrophic bacteria such as pseudomonas, spirillum, lactobacillus, bacillus, microaoccus, etc. For reduction to occur, the DO level must be near to zero, and carbon supply must be available to the bacteria. Because of low carbon content is required for the previous nitrification step, carbon must be added before de-nitrification can proceed. A small amount of primary effluent, bypassed around secondary and nitrification reactor can used to supply the carbon. However, the un-nitrified compounds in this water will be unaffected by the de-nitrification process and will appear in effluent. When complete nitrogen removal is required, an external source of carbon containing no nitrogen will be required. The most commonly used external source of nitrogen is methanol. 3 mg/L of methanol is

required for each milligram per liter of nitrate, when methanol added the reaction becomes to this:



2.6.1 Disinfection.

The last treatment step in a secondary plant is the addition of a dis-infectant to the treated wastewater. The addition of chlorine gas or some other form of chlorine is the process most commonly used for wastewater disinfection in the United State. Chlorine injected into the wastewater by automated feeding system. Wastewater then flows into a basin, where it held for about 15 minutes to allow the chlorine to react with the pathogens.

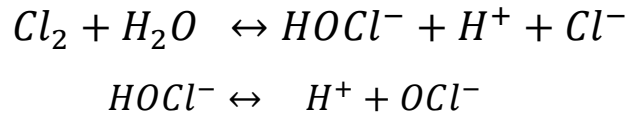
There is concern that wastewater disinfection may do more harm than good U.S. Environmental Protection Agency (USEPA) rules calling for disinfection to achieve 200 fecal coliforms per 100 mL of wastewater have modified to a requirement for disinfection only during the summer season when people may be exposed to contaminated water. There were three reasons for this change. The first was that the use of chlorine and, perhaps, ozone cause the formation of organic compounds that are carcinogenic. The second was the finding that the disinfection process was more effective in killing the predators to cysts and viruses than it was in killing the pathogens themselves. The net result was that the pathogens survived longer in the natural environment because there were fewer predators. The third reason was that chlorine is toxic to fish.

2.6.2 Factors affecting disinfection.

- Contact time
- pH
- Concentration or intensity of disinfectant.

- Concentration and type of microorganism.
- Concentration of interfering substances.

2.6.3 Chlorine Chemistry.

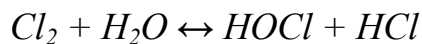


The pH affects this reaction; lower the pH drives the reaction to make HOCl. HOCl (hypo-chlorous acid) is more powerful and effective than OCl⁻ (hypochlorite ion). At a value of pH below 7.4 which conducive to HOCl formation. [15]

2.6.4 Chlorine Reactions with Ammonia.

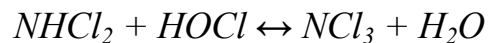
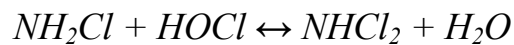
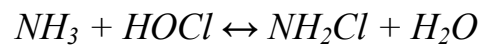
Chemistry of chloramines formation Previous researchers have studied the influences of pH, ammonia concentration, chlorine dosage, reaction time, as well as the precursor type and precursor concentration on the formation of chloramines. In aqueous systems, chlorine (Cl₂) reacts with ammonia (NH₃) to form chloramines. When chlorine added to water, it proceeds through a series of reactions. Chlorine will first react with organic materials and metals in the water, and it is not available for chlorination. This called the chlorine demand of the water. When chlorine used in the water treatment systems, it added in either a gaseous or a liquid form of chlorine.

Chlorine hydrolyzes very rapidly and produces hypo-chlorous acid (HOCl) and hydrochloric acid (HCl) as described below:



As pH varies in a system so does the concentration of hypo-chlorous acid versus the concentration of hypochlorite ion. Both HOCl and OCl⁻ are good disinfecting agents, but HOCl is more effective. Hypo-chlorous acid is a weak acid that ionizes at pH 7.5 and 25 °C (Lim Fang Yee et al) [16].

The hypo-chlorous acid will react with ammonia in the water and form chloramines, which as mentioned earlier are a benefit due to their strong residuals and because when chlorine converted to chloramines the formation of disinfection-by-products ceases. However, the chloramines are not very effective against viruses and are a weaker disinfectant. Hypo-chlorous acid reacts rapidly with ammonia to form mono-chloramine (NH_2Cl), dichloramine (NHCl_2) or nitrogen trichloride (NCl_3) as shown in reactions below:



The equations above simplified to illustrate the complex effect of the chlorine-to-ammonia ratio on chloramines formation [8].

The reactions are equilibrium indicating that both forward and reverse reactions occur to an appreciable extent. The reverse reactions also indicate that if chloramines dissolved in the water, chlorine and ammonia will generated. In the real-world sample, chloramines chemistry entails a complex series of reaction involving many species, pathway and mechanism [9].

In short, free chlorine, nitrogenous compound and chloramines chemically related and can converted into each other.

2.6.5 Forms of chlorine.

➤ Free available chlorine.

Free available chlorine defined as the concentration of chlorine existing in the form of hypo-chlorous acid and hypochlorite ions.

➤ Combined chlorine.

Combined chlorine is the concentration of chloro-amines formed when Non-nitrified effluent is chlorinated.

2.6.6 Design issues for chlorination.

- Chlorine dose and chlorine demand.
- Degree of mixing of chlorine: Good mixing at chlorine addition is more efficient and effective.
- Plug flow reactor: More efficient chlorination kinetics and lower coliform concentration.
- Detention time: Average detention time approximately from 15 to 30 minutes. One-hour detention time often recommended.

2.6.7 Chlorine Demand Chlorine Dose and Chlorine Residual.

The chlorine demand in mg/L varies according to the type of wastewater treatment method as shown on the following table:

$$\text{Chlorine dose} = \text{chlorine demand} + \text{chlorine residual}$$

Table (2.1): Chlorine Demand for Different Wastewater Effluents.

Wastewater treatment	Demand, mg/L
Primary effluent	10 to 25
Trickling filter effluent	5 to 15
Activated sludge effluent	4 to 10
Filtered activated sludge effluent	4 to 8
Nitrified effluent	4 to 8
Septic tank effluent	10 to 30
Intermittent sand filter effluent	2 to 6
Chlorine decay	2 to 4 mg/L-hr

2.6.8 Chlorine Injection to Treated Wastewater Effluent.

Chlorine added to the wastewater to satisfy all chemical demands. When these initial chemical demands have been satisfied, chlorine will react with substances

such as ammonia to produce chloramines and other substances, which although not as effective as chlorine, have disinfecting capability.

This produces a combined residual that can be measured using residual chlorine test methods. If additional chlorine is added, free residual chlorine can be produced.

Due to the chemicals typically found in wastewater, chlorine residuals are normally combined rather than free residuals.

Control of the disinfection process is usually based on maintaining total residual chlorine of at least 1.0 mg/L for a contact time of at least 30 minutes at design flow [18].

2.6.9 Primary Treatment (Primary Sedimentation Tanks).

The most types of these tanks usually fall into three categories:

- 1) Horizontal flow.
- 2) Solids contact.
- 3) Inclined surface.

The common types of horizontal flow clarifiers are rectangular, square, or circular shown on figure (2.20).

On the other hand, there are other types of sedimentation tanks such as surface tube settlers and parallel plate settlers.

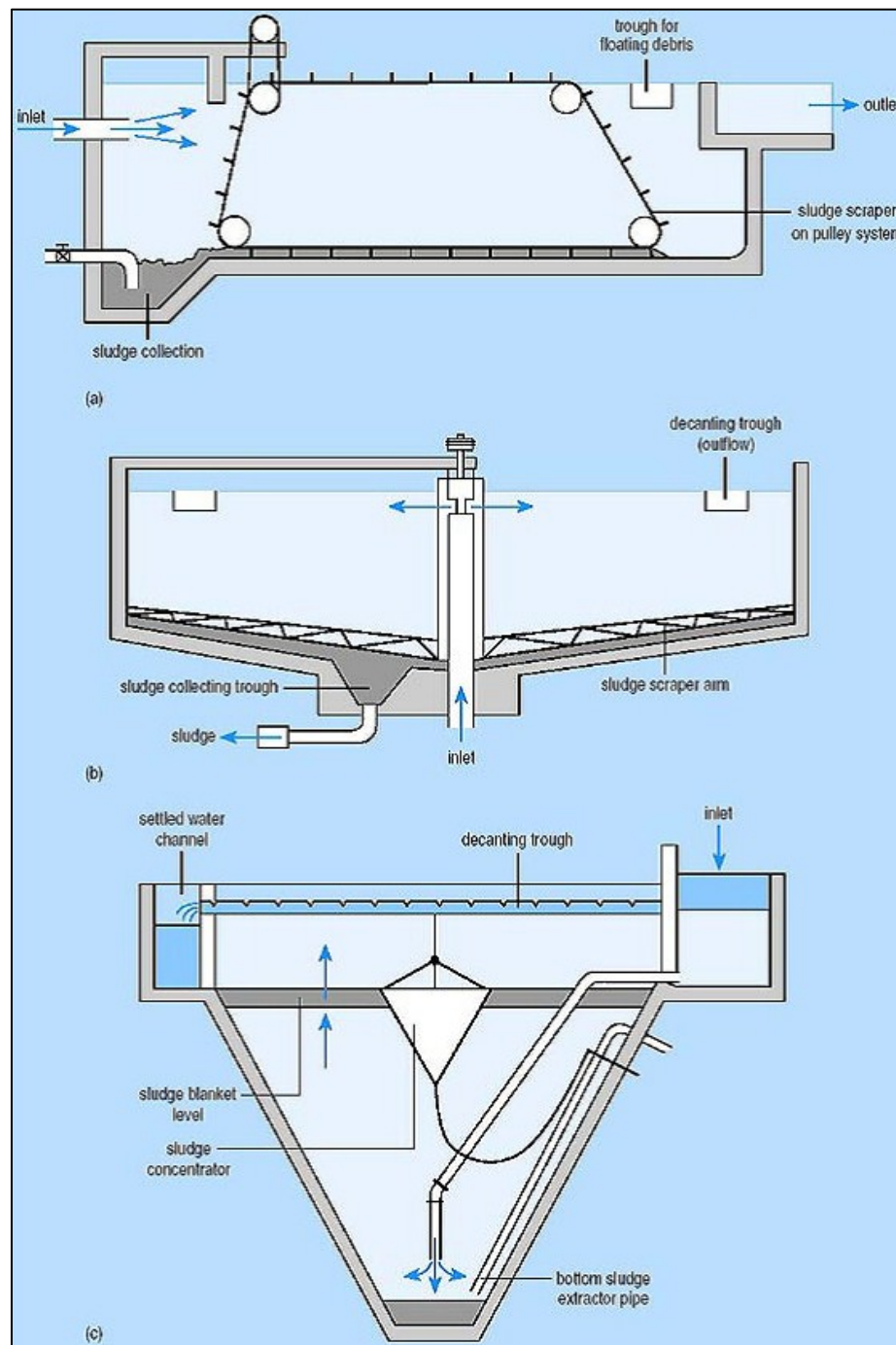


Figure (2.20): Typical Sedimentation Tanks:

(a) Rectangular Horizontal Flow Tank.

(b) Circular Radial-Flow Tank.

(c) Hopper-Bottomed Upward Flow Tank [4].

Chapter Three

Sludge Treatment

3.1 Sludge Treatment:

In the process of purifying the wastewater, another problem created sludge. The higher the degree of wastewater treatment, the larger amount of sludge that produced and must handle properly. The exception to this rule is where land applications or polishing lagoons are used. The proper treatment and disposal of the sludge can be complex and costly operation in a municipal wastewater treatment system. The sludge is made a materials settled from the raw wastewater and of solid generated in the wastewater treatment processes. The quantities of sludge involved are significant. For primary treatment, the amount of sludge may be in the range of 0.25 to 0.35 percent by volume of wastewater treated. When treatment upgraded to activated sludge, the quantities increase to 1.5 to 2 percent of this volume of water treated. The application of chemicals for phosphorus removal can add another 1% by volume of the treated wastewater. The sludge withdrawn from the treatment process is still largely water, as much as 97 percent. Sludge treatment processes, then are concerned with separating the large amount of water from the solid residues. The separated water returned to the wastewater plant for processing.

3.1.1 Activated Sludge Serves Several Purposes:

- 1) Reducing organic matter in wastewater by using a complex biological community in the presence of oxygen and converting the organic matter to new cell mass, carbon dioxide and energy.
- 2) Producing solids capable of bio-flocculating and settling out in the clarifier to produce and effluent low in Biochemical Oxygen Demand (BOD) and Total Suspended Solids (TSS).
- 3) Remove the organic matter (and N, P) from the wastewater by bacteria in suspension.

3.1.2 The Process of Activated Sludge:

A typical activated sludge plant identifies the three major control mechanism for the activated sludge treatment process:

- Aeration rate.
- Return sludge rate.
- Waste sludge rate.

In activated sludge process wastewater, containing organic matter aerated in an aeration basin in which microorganisms metabolize the suspended and soluble organic matter [20].

- Part of organic matter synthesized into new cells and part oxidized to CO_2 and water to derive energy.
- In activated sludge systems, the new cells formed in the reaction removed from the liquid stream in the form of a flocculent sludge in settling tanks.
- A part of this settled biomass, described as activated sludge returned to the aeration tank and the remaining forms waste or excess sludge.

The basic activated sludge process has several interrelated components. These components listed below:

- a) Aeration tank.
- b) Aeration source (air compressor).
- c) Secondary clarifier (SC).
- d) Recycle activated sludge (RAS).
- e) Waste Activated Sludge (WAS).

3.1.3 Aeration Tank.

A single tank or multiple tanks designed generally for either complete mix or plug flow with a detention time of as little as 2 hours and up to over 24 hours. The contents of the aeration tank referred to as mixed liquor.

- Aeration source: Generally, either diffused air or surface mechanical aeration used to supply oxygen and mix the aeration tank contents.
- Clarifier: A settling tank where the mixed liquor solids separated from the treated wastewater. Most treatment plants employ several secondary clarifiers.
- Recycle Activated Sludge: Solids that settle in the clarifier, and returned to the aeration tank.
- Waste Activated Sludge: Excess solids that must be remove from the system [11].

3.1.4 The Basic Processes for Sludge Treatment are as Flow:

1) Thickening:

Sludge thickening is the operation of separating water from sludge as much as possible by means of gravity or flotation.

2) Stabilization:

Converting the organic solids to more refractory (inert)forms so that they can be handled or used as soil conditioners without causing a nuisance or health hazard through processes referred to as “digestion.”(These are called biochemical oxidation process)”.

3) Conditioning:

Conditioning is the process of treating the sludge with a suitable chemical or heat so that the water can separated readily and easily.

4) Dewatering:

Dewatering or the removal of water associated with the sludge. This accomplished by Separating the water by subjecting the sludge to vacuum, pressure, or drying.

5) Reduction:

Reduction is the process of converting unstable solids to a stable form. This will accomplished by applying wet oxidation or incineration. (These methods are chemical oxidation processes); they decrease the volume of sludge, which will lead to solid reduction). Although a large number of alternatives are limited, the ultimate depository of the material contained in the sludge must either disposed through land, air, or water. Current policies discourage practices such as ocean dumping of sludge. Air pollution considerations necessitate “air pollution control facilities” as part of the sludge incineration process.

6) Sources and characteristics of sludge:

Before we begin the discussion of the various treatment processes, it is worthwhile to recapitulate the sources and nature of the sludge that must treated.

a) Grit:

The sand, broken glass, nuts, bolts, and other dense material that collected in the grit chamber is not true sludge in the sense that it is not fluid. However, it still requires disposal.

Because grit can drained of water easily and is relatively stable to biological activity, that is, it is not biodegradable; it generally trucked directly to a landfill without further treatment.

b) Primary or raw sludge:

Sludge from the bottom of the primary clarifiers contains from 3 to 8% solids (1% solids = 1g solids / 100 mL sludge volume), which is approximately 70 percent organic. This sludge rapidly becomes anaerobic and is highly odiferous.

c) Secondary sludge:

This sludge consists of microorganism and inert materials that have been wasted from the secondary treatment processes. Thus, the solids are about 90 percent organic. When the supply of air removed, this sludge also becomes anaerobic, creating noxious conditions if not treated before final disposal. The solids content depends on the source of wasted activated sludge, which is typically 0.5 to 2 % solids, while trickling filter sludge contains 2 to 5 % solids. In some case, secondary sludge, contain large quantities of chemical precipitates because the aeration tank used as the reaction basin for the addition of chemicals to remove phosphorus.

d) Mixed Liquor Suspended Solids MLSS:

The combination of raw sewage and biological mass commonly known as Mixed Liquor.

- In all activated sludge plants, once the sewage (or industrial wastewater) has received sufficient treatment, excess mixed liquor discharged into settling tanks and the treated supernatant run off to undergo further treatment before discharge.
- Part of the settled material, the sludge (RAS) x returned to the head of the aeration system to re-seed the new sewage, entering the tank.
- Mixed Liquor is a mixture of raw or settled wastewater and activated sludge within an aeration tank in the activated sludge process. Mixed

Liquor Suspended Solids (MLSS) is the concentration of suspended solids in the mixed liquor, usually expressed in milligrams per liter (mg/l).

- If MLSS content is too high, the process is prone to bulking and the treatment system becomes overloaded. This can cause the dissolved oxygen content to drop with the effect that organic matters are not fully degraded and biological 'die off' – Excessive aeration required which wastes electricity if MLSS content is too low, The process is not operating efficiently and is wasting energy. Typical Control band 2,000 to 4,000 mg/l.

7) RAS and WAS:

A proportion of the floc called Return Activated Sludge (R.A.S.) and used to re-seed the process.

- Measuring the solids concentration of RAS allows the return volume to be adjusted to keep the solids level in the aeration basin within the control parameters.
- Excess sludge which eventually accumulates beyond that returned is defined as Surplus or Waste Activated Sludge (SAS/WAS).
- This is removed from the treatment process to keep the ratio of biomass to food supplied (sewage or wastewater) in balance. Typical Range – 4,000 to 6,000 mg/l [12].

3.2 Tertiary Sludge.

The characteristics of sludge from the tertiary treatment processes depend on the nature of the process. For example, phosphorus removal result in a chemical sludge that is difficult to handle and treat. When phosphorus removal occurs in the activated sludge process, this chemical sludge combined with the biological

sludge, making the latter more difficult to treat. Nitrogen removal by denitrification results in a biological sludge with properties very similar to those of waste-activated sludge.

3.3 Physical and Chemical Process.

3.3.1 Coagulation:

Coagulation is the process by which colloidal particles and very fine solid suspensions initially present in a wastewater combined into larger agglomerates that can be separated via sedimentation, flocculation, filtration, centrifugation or other separation methods. Commonly achieved by adding different types of chemicals (coagulants) to the wastewater to promote destabilization of the colloid dispersion and agglomeration of the resulting individual colloidal particles. Indicates the process through which colloidal particles and very fine solid suspensions are destabilized so that they can begin to agglomerate.

3.3.2 Flocculation:

Refers to the process by which destabilized particles actually conglomerate into larger aggregates so that they can be separated from the wastewater. Furthermore, that the primary particles induced to approach close enough together, make contact, and progressively form larger agglomerates, or flocs. The principal parameter governing the rate of flocculation is the applied velocity gradients and time of flocculation. These two parameters influence the rate and extent of particle aggregation and the rate and extent of breakup of these aggregates. Velocity gradient applied. The degree or extent of flocculation governed by both applied velocity gradients and time of flocculation. These two parameters influence the rate and extent of particle aggregation and the rate and extent of breakup of these aggregates [10].

Chapter Four

Calculation and Result

4.1 Design of Wastewater Treatment Plant.

4.1.1 General:

The procedures and stages of design fixed by the standards and assumptions according to reference that considered.

The treatment on this plant depend specifically on activated sludge process.

Stages of design the plant as the following:

- Screen.
- Grit chamber.
- Primary sedimentation tank.
- Extended aeration tank.
- Secondary sedimentation tank.
- Nitrification.
- Disinfection tank.

4.2 Calculations & Results.

4.2.1 Introduction.

Step 1: determination of average and peak flow rates:

This project include designing of a wastewater treatment plant for a residential community with a population of (75,000 person), and with a design period of (20 years).

$P = 75,000$ people.

$t = 20$ years.

Population rate constant $k = 0.03$.

Assume the return amount of wastewater = 75 %.

$P_t = P + (K * P_{20})$.

$P_{20} = 75,000 + (0.03 \times 20 \times 75,000)$.

$P_{20} = 120,000$ person.

Daily wastewater share / person = $620 \times 0.75 = 465$ ℓ/day. Person.

$$\text{Average flow} = \frac{(465 \times 12,0000)}{1,000} = 55,800 \text{ m}^3/\text{day}.$$

$$\text{Net average flow} = \frac{55,800}{86,400 \times 0.4} = 1.61 \text{ m}^3/\text{sec}.$$

Calculation for the ratio of the maximum sewage flow to the average (M).

$$M = 1 + \frac{14}{4 + 120^{0.5}}.$$

$$M = 1.93 .$$

Say: $M = 2$.

Peak flow rate = $M \times$ average flow.

$$= 2 \times 1.61 = 3.22 \text{ m}^3/\text{sec}.$$

4.2.2 Design of Screen:

Design Criteria Used:

- Velocity through rack at max flow = 0.9 m/sec.
- Bar spacing (clear) = 2.5 cm.
- Provide two identical barracks for max flow = 2.
- Screen angle $\theta = 75$.
- The wide of each bar = 1cm, Max. Flow = 3.22 m³/sec.
- Average flow = 1.61 m³/sec.

Step 1: Compute of dimension of screen chamber.

Assume that the depth of the flow in the rack chamber = 1.18 m.

Clear area through = $Q_{\text{aver}} / \text{Velocity through rack}$.

$$\text{The rack} = \frac{1.61}{0.92} = 1.79 \text{ m}^2$$

$$\text{Clear width} = \text{Area} / \text{Depth} = \frac{1.75}{1.18} = 1.52 \text{ m}.$$

$$\text{No. of spacing} = \frac{1,500 \text{ mm}}{25 \text{ mm}} = 60.8 \text{ space.}$$

$$\text{Total no. of bars} = 60.8 - 1 = 59 \text{ bar.}$$

Provide bars with 10 mm width.

$$\text{Width of the chamber} = 1.5 + \frac{(10 \times 60)}{1,000} = 2.1 \text{ m.}$$

$$\text{Efficiency} = \frac{\text{number of spacing} \times \text{clear spacing}}{\text{width of rack chamber}}$$

$$\text{Screen efficiency} = 25 \times \frac{60}{2,100} = 0.714 \times 100 = 71.4 \text{ \%}.$$

Step 2: Determinations of head loss across screens.

Equation (1) used to calculate head loss through clean screen only bars.

$$h_L = \beta \left(\left(\frac{W}{B} \right)^{4/3} \right) (h(r) \sin(\theta)) \quad (1)$$

Equation (2) used to calculate head loss through clean or partly clogged.

$$h_L = \left(\frac{1}{0.7} \right) \times \left((V_v)^2 \left(\frac{2}{2g} \right) \right) \quad (2)$$

Where:

H_L = Head loss through the rack, m.

V_v = Velocity through the rack and in the channel upstream of the rack, (0.5 m/sec).

g = Gravity Acceleration = 9.81 m/s²; w = Maximum width of the bar = 10 mm.

b = Minimum clear spacing of bars = 100 mm.

h_r = Velocity head of the flow approaching the bars = $[(V_v)^2 / 2g]$.

θ = Angle of bars with horizontal.

β = Bar shape factor = 2.42.

Case one: when the screen is clean:

$$Hl = 2.42 \times \frac{10^{\frac{4}{3}}}{100} \times 0.025 \times \sin(75).$$

$$Hl = 0.0027 \text{ m.}$$

$$\text{Clean or partially clogged screen (hL), m} = \frac{0.9^2 - 0.5^2}{2 \times 9.81 \times 0.7} = 0.04077 \text{ m.}$$

4.2.3 Design of the Aerated Grit Chambers.

Geometry of Grit Chamber.

Number of Grit Chambers = 3.

Detention time (τ) = 4 min.

Provide Avg. water depth at mid width = 3.65 m.

Provide free board = 0.8 m.

Provide L: W = 4.

Step 1: Max design flow through each chamber.

$$= \frac{3.22}{3} = 1.073 \text{ m}^3/\text{sec.}$$

Step 2: Volume of each chamber.

$$= 1.073 \times 4 \times 60 = 257.5 \text{ m}^3.$$

Step 3: Total depth of grit chamber.

$$L T = 3.65 + 0.8 = 4.45 \text{ m.}$$

Step 4: Surface area of chamber: $= \frac{99.8 \text{ m}^3}{3.65 \text{ m}} = 27.3 \text{ m}^2$.

Length to width ratio = 4:1 $\Rightarrow$ area = 4 W².

Step 5: Width of the chamber = 4.28 m.

Length of the chamber, L = 17.15 m.

4.2.4 Design of the Aerated Grit Chambers Air Supply System.

Air supply rate per chamber length = 7.8 Lit / (sec-m).

Provide 150% capacity for peaking = 150.

Step 6: Theoretical air required per chamber: $= 7.8 \times 17.15 = 133.77 \text{ l/s}$.

Step 7: Total capacity of the diffusers; $= 1.5 \times 133.77 = 152.1 \text{ l/s per chamber}$.

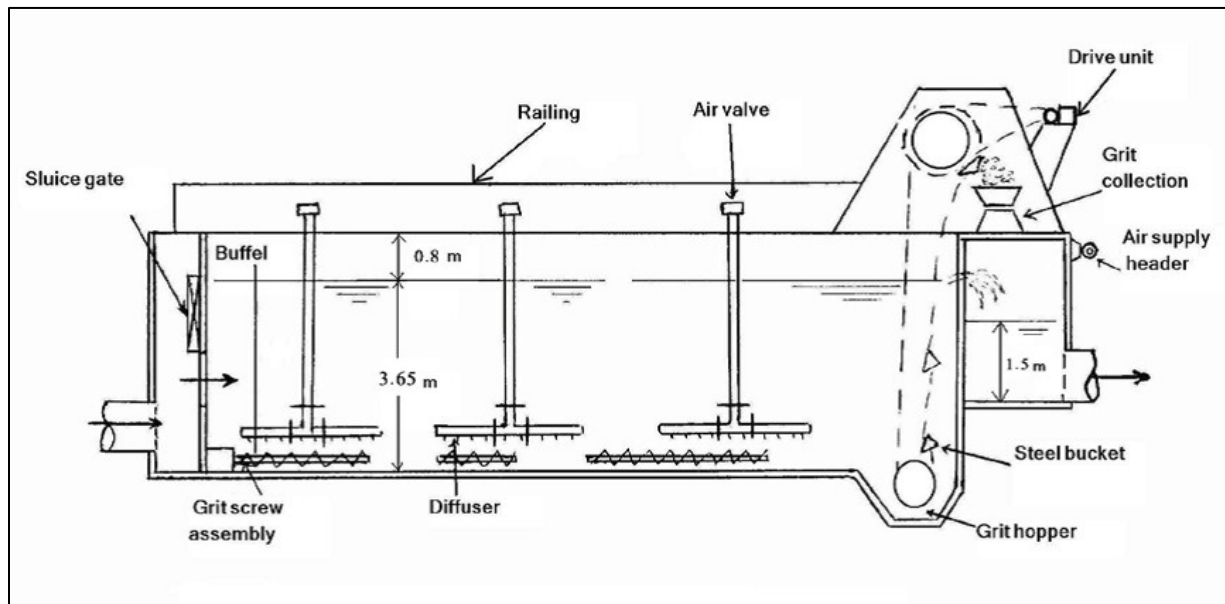


Figure (4.1): Design Details of the Aerated Grit Chamber.

4.2.5 Design Calculations for Primary Sedimentation Tanks.

Design Critter Used:

- Number of sedimentation = 8.

- Overflow rate less than $= 36 \text{ m}^3 / (\text{m}^2\text{-d})$.
- Detention time should not be less than $= 1.5 \text{ h}$.
- Weir loading shall be less than $186 \text{ m}^3 / (\text{m-d}) = 186$.
- Liquid depth in basin shall be no less than $= 2 \text{ m}^2$.
- Influent $\text{BOD}_5 = 250 \text{ mg / l}$.
- Influent $\text{TSS} = 260 \text{ mg / l}$.
- Overflow rate $= 36 \text{ m}^3 / (\text{m}^2\text{-d})$.
- Provide free board $= 0.6 \text{ m}$.

Step 1: Average design flow through each basin; $= \frac{1.61}{8} = 0.2018 \text{ m}^3/\text{sec}$.

Step 2: Surface area $= \frac{0.2018 \times 86400}{36} = 484.38 \text{ m}^3 / \text{m}^2\text{-day}$.

Step 3: Use length to width ratio (4:1) $\rightarrow A = L \times W = 4 W^2$.

So, wide of each basin $= 11.5 \text{ m}$.

Length of each basin $= 4 \times 11.5 = 44.02 \text{ m}$.

Provide average water depth at mid. length of the tank $= 3.1 \text{ m}$.

Average depth of the basin $= 3.1 + 0.6 = 3.7 \text{ m}$.

Step 4: Check Overflow Rate.

Overflow rate at $= \frac{0.2018 \times 86400}{44 \times 11} = 36 \text{ m}^3 / \text{m}^2\text{-d}$.

Max Flow $= 3.229 \text{ m}^3 / \text{sec}$.

Max Flow for each tank $= \frac{3.229}{8} = 0.404 \text{ m}^3 / \text{sec}$.

Overage rate at $= \frac{0.404 \times 86400}{44 \times 11} = 72 \text{ m}^3 / \text{m}^2\text{-d}$.

Step 5: Detention Time.

Average volume of the basin = $3.1 \times 44 \times 11 = 1501.56 \text{ m}^3$.

Detention time = $\frac{1501.56}{0.2018 \times 3600} = 2.07 \text{ hr.}$

Detention time at MAX design flow = $\frac{1501.56}{0.404 \times 3600} = 1.03 \text{ hr.}$

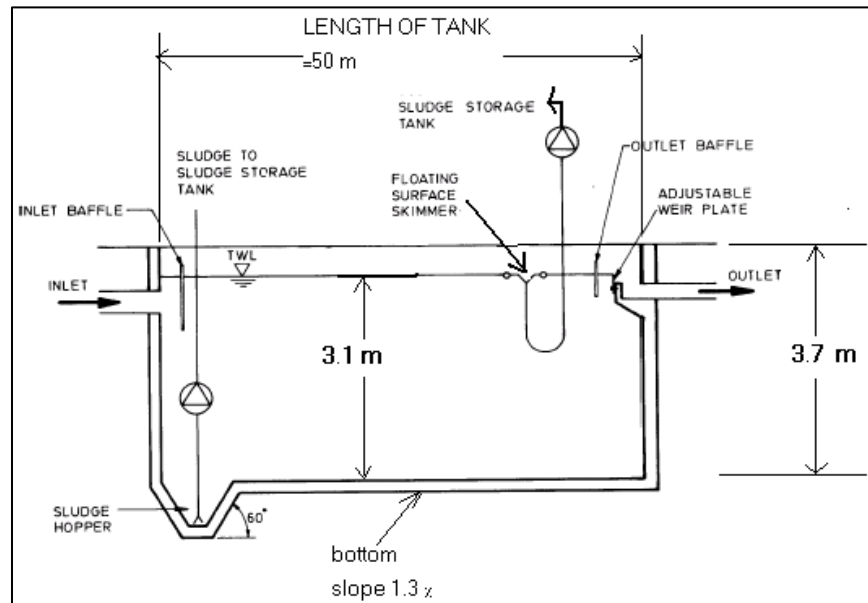


Figure (4.2): Design Calculations for the Biological Reactor.

4.2.6 Design of Biological Reactor.

Avg. design flow rate = $1.615 \text{ m}^3 / \text{sec.}$

Peak flow rate = $3.229 \text{ m}^3 / \text{sec.}$

Row ww $\text{BOD}_5 = 250 \text{ mg / L.}$

Row WW TSS = 260 mg / L.

Effluent $\text{BOD}_5 = 20 \text{ mg / L.}$

Effluent TSS = 24 mg / L.

WW Temperature = 20 C.

4.2.7 Operational Parameters and Biological Kinetic Coefficients.

MCRT = 10 days.

MLVSS = 3,000 mg / L.

Ratio VSS / TSS = 0.8.

Ratio BOD₅ / BODU = 0.67.

TSS in RAS, mg / L = 1,500.

Y, mg VSS / mg BOD = 0.5.

Kd = 0.06 d⁻¹.

Step 1: Plant BOD and TSS loading calculations.

Design flow (Q) = $1.615 \times 68,400 = 139,500 \text{ m}^3 / \text{d}$.

BOD loading, Kg / d = $\frac{250}{1000} \times 139,500 = 34,875 \text{ kg} / \text{d}$.

TSS loading, Kg / d = $\frac{260}{1,000} \times 139,500 = 36,270 \text{ kg} / \text{d}$.

Step 2: Primary sludge BOD and TSS calculations.

% BOD Removal in PC = 33.

% TSS Removal in PC = 67.

Specific Gravity of Prim Sludge = 1.05.

Wt % solids in prim sludge = 4.40.

BOD removed = $\frac{34,875 \times 33}{100} = 11,508.7 \text{ Kg} / \text{d}$.

$$\text{TSS removed} = \frac{36,270 \times 67}{100} = 24,300.9 \text{ Kg / d.}$$

$$\text{Sludge flow rate} = \frac{24,300.90}{1,000 \times 1.05} \div 0.044 = 525.99 \text{ m}^3 / \text{d.}$$

Step3: Secondary treatment influent BOD and TSS.

$$\text{Effluent flow from PC} = 139,500 - 525.99 = 138,974.01 \text{ m}^3 / \text{d.}$$

$$\text{Effluent BOD from PC} = 34,875 - 11,508.7 = 23,366.25 \text{ kg / d.}$$

$$\text{Influent BOD to Secondary treat (S}_0\text{)} = \frac{23,366.25 \times 1,000}{13,8974.01} = 168.13 \text{ mg / L.}$$

$$\text{TSS leaving PC} = 326,270 - 24,300.9 = 11969.10 \text{ mg / L.}$$

$$\text{Influent TSS to Secondary treat} = \frac{11,969.1 \times 1,000}{13,8974.01} = 86.12 \text{ mg / L.}$$

Step 4: Estimate the soluble BOD₅ escaping treatment S, in the effluent.

Effluent BOD = influent soluble BOD escaping treatment, S + BOD of effluent suspended solids.

Determine the BOD₅ of the effluent (assuming 63% biodegradable).

$$\text{Biodegradable effluent solids} = \frac{20 \times 63}{100} = 12.60 \text{ mg / l.}$$

$$\text{Oxygen consumption mg O}_2 / \text{mg cells} = 1.42 \text{ O}_2 / \text{mg cell.}$$

$$\text{Ultimate Biodegradable effluent solids} = 12.60 \times 1.42 = 17.89 \text{ mg / l.}$$

$$\text{Influent soluble BOD}_5 \text{ escaping treatment, mg / L} = 20 - 17.89.$$

$$S = 8.01 \text{ mg / l.}$$

Step 5: Calculating treatment efficiencies.

The efficiency of biological treatment based on soluble BOD is:

$$E = \frac{S_o - S}{S_o} \times 100 \quad (3)$$

$$\text{Efficiency } (\eta) = \frac{168.13 - 8.01}{168.13} \times 100 = 95.23 \%$$

$$\text{Overall plant efficiency, } E = \frac{250 - 20}{250} \times 100 = 92 \%$$

Step 6: calculating the volume of the biological reactor.

$$V = \frac{\theta_c Q Y (S_o - S)}{X(1 + K_d \theta_c)} \quad (4)$$

Where:

V = Volume of aeration basin, m³.

θ_c = Mean cell residence time based on solids in the aeration basin, day.

Q = Influent wastewater flow rate, m³ / d.

Y = Yield coefficient over finite period of log growth, g / g.

S_o = Influent soluble BOD₅ concentration mg / ℓ.

S = Effluent soluble BOD₅ concentration mg / ℓ.

X = Concentration of MLVSS maintained in the aeration basin mg/ℓ (g / m³).

K_d = Endogenous decay coefficient, d⁻¹.

$\Delta x / \Delta t$ = Growth of biological sludge over time period Δt , mg / ℓ (g / m³).

Q_r = Waste sludge flow rate from the sludge return line, m³ / d.

X_r = Concentration of sludge in the return sludge line, mg / ℓ (g / m³).

Q_w = Waste sludge flow rate from aeration tank, m³ / d.

Design Criteria Used:

* Return sludge concentration (X_r) = 15,000 mg / ℓ (TSS).

* BOD₅ for the effluent (SS) = 0.63.

Influent BODR₅ R and TSS = 200 and 150 (mg / ℓ) respectively.

Average flow = 1.61 m³ / s = 139,104 m³ / day.

$$\text{Since: } V = \frac{Q Q_c Y (S_o - S)}{X (1 + k_d Q_c)} = \frac{13 \cdot 10 \cdot 0.5 (200 - 8)}{3000 (1 + 0.06 - 1 \cdot 10)}$$

$$V = 23,179.94 \text{ m}^3.$$

Step 7: Dimensions of aeration basin.

Assume number of tanks = 4.

Assume L/W ratio = 2.

Water depth = 4.40 m.

Volume for each basin = 23,179.94 m³.

Free board depth = 0.6 m.

$$\text{Cross-sectional area of rectangle tank} = \frac{23,179.49}{4 \times 4.40} = 1,317.04 \text{ m}^2.$$

$$(W)(L) = (W) (2W) = \text{area.}$$

$$\text{Width (W)} = \left(\frac{1,317.04}{2} \right)^{0.5} = 25.66 \text{ m.}$$

$$\text{Length (L)} = 2 W = 25.66 \times 2 = 51.32 \text{ m.}$$

$$\text{Total tank depth} = 4.40 + 0.6 \rightarrow d = 5 \text{ m.}$$

Step 8: determination of the amount of sludge wasted.

Calculations for the detention time:

$$\theta_c = \frac{VX}{Q_w aX + Q_e X_e} \quad (5)$$

Where:

Q_w = flow of waste sludge from return sludge line, m^3 / d .

X = microorganism concentration in return sludge line, mg / L .

Q_e = flow of treated effluent, m^3 / d .

X_e = microorganism concentration (VSS) in effluent, mg / L .

$$MLSS (X) \text{ mg / L} = \frac{3,000}{0.8} = 3,750 \text{ mg/l.}$$

$$X_e = 24 \times 0.8 = 19.20 \text{ mg / l.}$$

$$\text{Using Eq. (5), } QW = \frac{3,000 \times 23,179.94 - 23,179.94 \times 10 \times 19.20}{3,750 \times 10}.$$

$$QW = 1,142.85 \text{ m}^3 / d = 0.0132 \text{ m}^3 / \text{sec.}$$

$$\text{Amount of SS sludge produced} = \frac{3,000 \times 1,000 \times 27,179.94}{1,000,000 \times 10} = 6,953.98 \text{ kg / d.}$$

$$\text{Amount of VSS sludge produced} = \frac{6,953.98}{0.8} = 8,692.48 \text{ kg / d.}$$

$$QW = \frac{8,692.48 \times 1,000,000}{15,000 \times 1,000} = 579.50 \text{ m}^3 / d = 0.00671 \text{ m}^3 / \text{sec.}$$

Step 9: Estimate the quantity of sludge to be waste daily.

$$Y_{obs} = \frac{Y}{1 + K_d \theta_c} \quad (6)$$

$$\text{Observed yield } (Y_{obs}) = \frac{0.5}{1 + 0.06 \times 10} = 0.313.$$

Step 10: Calculating the increase in mass of MLVSS, (P_x).

$$p_x = Y_{obs} Q (S_o - S) \times (1 \text{ kg} / 1,000 \text{ g}) \quad (7)$$

$$P_x = \frac{1.608 \times 138,974.006 \times (168.134 - 8.02)}{1,000} = 6,953.981 \text{ Kg} / \text{d}.$$

Calculating the increase in mass of MLSS (TSS), P_{ss} .

$$P_{ss} = \frac{6,953.981}{0.8} = 8,692.477 \text{ kg} / \text{d}.$$

Calculating the TSS lost in the effluent, P_e .

$$P_e = (Q - Q_{wa}) \times \text{TSS out} / 1,000 = \frac{(13,8974.006 - 1,142.85) \times 24}{1,000}.$$

$$P_e = 3,307.948 \text{ kg} / \text{d}.$$

Step 10: Calculating the amount of sludge to be wasted.

$$\text{Waste sludge} = P_{ss} - P_e = 8,692.4 - 3,307.948 = 5,384.53 \text{ kg} / \text{d}.$$

Step 11: estimating the flow of sludge return (Q_r).

$$Q_r = \frac{QX}{(X_r - X)} = \frac{13,8974.01 \times 3,000}{15,000 - 3,000} = 34,743.5 \text{ m}^3 / \text{d} = 0.4021 \text{ m}^3 / \text{sec}.$$

$$\text{Reflux ratio } (Q_r / Q) = \frac{3,4743.5}{13,8974.01} = 0.25.$$

If assume number of basin = 8.

$$\text{So, } Q_r \text{ for each basin, } \text{m}^3 / \text{d} = \frac{3,4743.5}{8} = 4,342.94 \text{ m}^3 / \text{d} = 0.0503 \text{ m}^3 / \text{sec}.$$

Step 12: Check hydraulic retention time ($HRT = \theta$).

$$\theta = \frac{V}{Q} = \frac{23,179.94}{19,8974.01} = 0.1668 \text{ days} = 4 \text{ hr}.$$

The preferred range of HRT is 5 – 15 h. (from appendix A).

Step 13: Determination of the F/M ratio using (U).

$$E = \frac{S_o - S}{\theta X} \quad (8)$$

$$U = \frac{168.134 - 8.012}{0.1668 - 3,000} = 0.3200 \text{ d}^{-1}.$$

$$F / M \text{ Ratio} = \frac{0.3200}{0.92} = 0.34783.$$

Step 14: Check Organic loading and mass BODu utilized.

$$E = \frac{Q S_o}{V} \quad (9)$$

$$\text{Loading} = \frac{13,8974.01 \times 168.134}{23,179.94} = 1.008 \text{ kg BOD}_5 / (\text{m}^3 \cdot \text{d}).$$

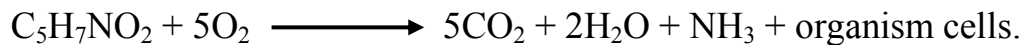
$$\text{BODu} = Q (S_o - S) / (\text{Ratio BOD}_5 / \text{BODu}).$$

$$\text{BODU} = \frac{13,8974.01 \times (168.134 - 8.012)}{0.67} = 33,213.045 \text{ kg / d.}$$

Step 15: Compute Theoretical oxygen requirements.

$$O_2 = \frac{Q(S_o - S)}{\left(1000 \frac{g}{kg}\right) f} - 1.42 p_x \quad (9)$$

The biochemical reaction can expressed as below:



$$113 \quad 5 \times 32 = 160$$

$$1.42$$

$$O_2 = \frac{138,974.01 \times (168.13 - 8.012)}{\left(\frac{1,000}{0.67}\right)} - 1.42 \times 6,954 = 23,338.392 \text{ Kg O}_2 / \text{d.}$$

Step 16: Compute the volume of air required.

$$\text{Density of AIR} = 1.202 \text{ Kg / m}^3.$$

Weight % oxygen in air = 23.3.

Air blower efficiency (%) = 8.

Safety factor (dim) for actual air = 2.

Theoretical AIR required = $\frac{23,338.392}{1.202 \times 0.233} = 83,690.945 \text{ kg / d.}$

Actual Air required = $\frac{83,690.945}{0.08} = 1,046,136.810 \text{ m}^3 / \text{d} = 726.484 \text{ m}^3 / \text{min.}$

The design air required (with a factor of safety 2) is:

Air = $726.484 \times 2 = 1,452.968 \text{ m}^3 / \text{min.}$

Step 17: Check the volume of air required per BOD₅ removed and per unit volume of wastewater and aeration tank.

Air supplied / Kg BOD₅ removed:

$$\text{air} = \frac{1046136.8 \times 1000}{138974.01 \times (168.13 - 8.012)} = 47.012 \text{ m}^3 \text{ of air / Kg of BOD}_5$$

Air supplied / m³ wastewater treated:

$$= \frac{1,046,136.8}{138,974.01} = 7.528 \text{ m}^3 \text{ of air / m}^3 \text{ of wastewater.}$$

Air supplied / m³ aeration tank volume.

$$= \frac{1,046,136.8}{23,179.94} = 45.131 \text{ m}^3 \text{ of air / m}^3 \text{ tank volume.}$$

4.2.8 Design Calculations for secondary Clarifiers.

Design Criteria used:

Provide this number of circular clarifiers = 8.

The overflow rates at avg. flow conditions less than = 15.

The overflow rates at peak flow conditions less than = 40.

Step 1: Design flow to Secondary clarifiers.

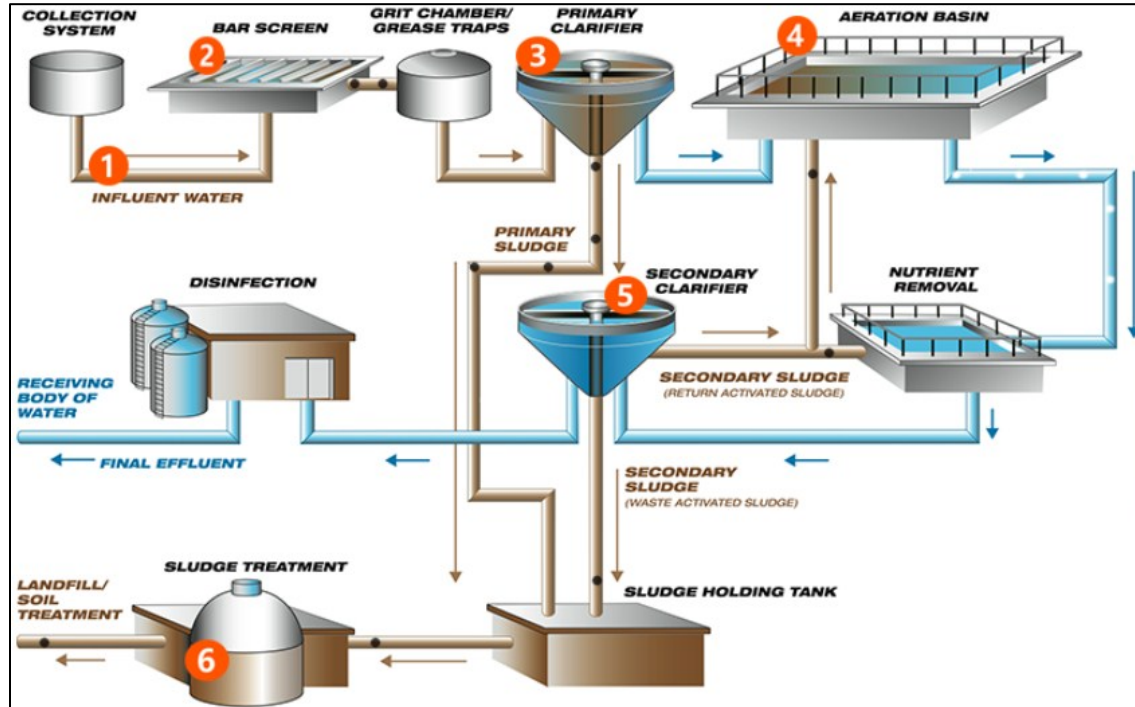


Figure (4.3): Design Flow to Secondary Clarifiers.

$$SC = Q_{avg} + Q_r - Q_w = 1.60850 + 0.05027 - 0.00671 = 1.65205 \text{ m}^3 / \text{sec}.$$

Design flow for each SC with no. of circular clarifiers = 8.

$$SC = 1.65205 / 8 = 0.20651 \text{ m}^3 / \text{sec}.$$

Assume surface flow rate (SOR) = 15 m / d.

$$\text{Top area of SC} = \frac{0.20651 \times 86,400}{15} = 1,189.47871 \text{ m}^2.$$

$$\text{SC diameter} = \sqrt{\frac{4 \times 1,189.4787}{3.14}} = 38.91642 \text{ m}.$$

$$\text{Actual Area} = \left(\frac{3.14 \times 38.91642}{4} \right)^{0.5} = 1,189.47871 \text{ m}^2.$$

Step 3: Check the surface overflow rate at peak design flow.

$$\text{Peak flow rate} = \frac{0.20651 \times 86,400}{1,189.47871} = 15 \text{ m}^3 / \text{d}.$$

$$\text{Total peak design flow plus } Q_r = 3.2917 + 0.05027 = 3.27943 \text{ m}^3 / \text{sec}.$$

$$\text{At peak design flow plus } Q_r \text{ for each} = 3.27943 / 8 = 0.40993 \text{ m}^3 / \text{sec}.$$

$$\text{Overflow rate} = \frac{0.40993 \times 86,400}{1,189.47871} = 29 \text{ m}^3 / \text{m}^2 - \text{d (satisfy)}.$$

Step 4: Depth of secondary clarifier.

Provide average side water depth = 3.5 m.

For additional safety provide a free board= 0.5 m.

$$\text{Total depth of clarifier} = 3.5 + 0.5 = 4 \text{ m}.$$

$$\text{Average Volume of SC} = 4 \times 1,189.48 = 4,757.91 \text{ m}^3.$$

Detention time under average design flow plus recirculation:

$$= \frac{4,757.91}{0.40993 \times 3,600} = 3.22 \text{ hr}.$$

Detention time under emergency condition (peak design flow plus recirculation when one clarifier is out of service):

$$= \frac{4,757.91}{0.46849 \times 3,600} = 2.82 \text{ hr}.$$

Step 5: Sludge calculations produced from PC and aeration basins.

Sludge produced from primary sedimentation:

$$\text{Specific gravity of sludge} = 1.03 \text{ g} / \text{cm}^3.$$

Typical solids content = 4.5 percent.

Amount of solids produced per basin per day at a removal rate of 63 percent

$$* s = \frac{260 \times 0.63 \times 0.2018 \times 86,400}{1,000} = 2,856.26 \text{ kg / d.}$$

Average quantity of sludge produced per day from the eight basins:

$$= 8 \times 2,856.26 = 22,850.10 \text{ kg.}$$

Volume of sludge produced from PCs, m³ / day.

$$= \frac{22,850.10}{1.03 \times 0.001 \times 0.0450 \times 100^3} = 492.99 \frac{m^3}{d}.$$

Volume of sludge produced from aeration:

$$= \frac{63,953.98}{1.03 * 0.001 * 0.0450 * 100^3} = 6,953.98 \frac{m^3}{day}.$$

Total of sludge (PCs and Area basins) = 22,850.10 + 6,953.98 = 29,804.08 Kg/d.

Total daily volume of sludge = 492.99 + 29,804.08 = 643.02 m³ / d.

Step 5: design calculations of gravity thickener:

Sludge at specific gravity = 1.03.

Drying ratio (wt %) = 5.

Number of gravity thickeners = 6.

Total flow to sludge gravity thickeners:

$$= \frac{29,804.08}{\left(\frac{1.03}{1,000}\right) * 0.05 * 100^3} = 578.72 \frac{m^3}{d}.$$

$$\text{Flow to each sludge thickener} = \frac{587.72}{6} = 96.45 \frac{\text{m}^3}{\text{d}}.$$

Step 6: Design Calculations for Drying Beds:

Assume the depth of sludge over drying bed = 0.25 m.

Total volume of sludge = 643.02 m³ / day.

$$\text{Surface area} = \frac{643.02}{0.25} = 2,572.09 \text{ m}^2 / \text{d}.$$

Provide seven drying beds.

$$\text{Surface area for each bed} = \frac{2,572.09}{6} = 3,626.5 \text{ m}^2 / 6 = 428.68 \text{ m}^2 / \text{d}.$$

Assume length of each bed = 30 m.

The width of each bed = 428.68 / 30 = 14.29 m.

Actual area for one bed = 30 × 14.29 = 428.68 m² / d.

Area need per week = 7 × 428.68 = 3,000.77 m².

Step 7: anaerobic digester design.

S.G = 1.120.

MCRT (θ_c) = 10 days.

Organic content in raw sludge wt % = 70.

I S.G inert content in raw sludge Wt % = 30.

S.G after digestion = 1.050.

Required digester volume = 10 × 643 = 6,430.223 m³.

Surface area (A_s) = V / H = 6,430.223 / 15 = 23.36 m².

Use Two Digesters.

$$Q_t = 29,804.1 / 2 = 321.51 \text{ m}^3 / \text{d}.$$

Volume for each digester, $\text{m}^3 = Q_t (\text{new}) \text{ MCRT}.$

$$Q_t = 321.51 \times 10 = 3,215.1.$$

$$\text{Surface area (As)} = V / H, \text{ m}^2 = 3,215.1 / 2 = 214.3.$$

Digester diameter = 17.

$$\text{Surface area (As)} = \frac{3.14 \times 16.8 \times 16.8}{4} = 222.2 \text{ m}^2.$$

$$\text{Correct H for new} = \frac{3,215.1}{222.2} = 14.47.$$

Step 8: Anaerobic digester design.

$$R_s = 13 \times \text{Ln} [(\theta_c) + 18.9] \quad (10)$$

$$\text{Ln} (\theta_c) = 2.30$$

$$\text{Efficiency of solid reduction (Rs)} = 13.7 \times 2.30 + 18.9 = 50.4.$$

$$\text{Amount of thickens sludge per digester } M_t = \frac{2,9804.1}{2} = 14,902.0 \text{ Kg} / \text{d}.$$

$$\text{Amount of Organic matter for each digester} = \frac{14,902 \times 70}{100} = 4,470.6 \text{ Kg} / \text{d}.$$

Approx. % of CH_4 produced from total gas = 60.

$$\text{Amount of digested sludge (M digest)} = \frac{10,431.4 \times 60}{100} = 6,258.9 \text{ Kg} / \text{d}.$$

$$\text{m}^3 \text{ of total gas produced} / \text{Kg digested sludge} = 0.5.$$

$$\text{Total Gas produced from anaerobic digestion} = 0.5 \times 6,258.9 = 3,129.4 \text{ m}^3 / \text{d}.$$

$$\text{CH}_4 \text{ produced for each digester, m}^3 / \text{d at 1 atm \& 0}^\circ\text{C} = 1,877.7.$$

Storage pressure of $\text{CH}_4 = 2 \text{ atm}.$

Volume of CH₄ storage tank m³ at 2 atm $(V_1 \times P_1 / P_2) = \frac{1,877.7 \times 1}{2} = 93.8$.

Amount of destroyed organic matter (Md) = $\frac{10,431.4 \times 60}{100} = 6,258.9$ kg / d.

Mass of inert material = $10,431.4 - 6,258.9 = 4,172.6$ Kg / d.

Total inert material leaving digester = $4,470.6 + 4,172.6 = 8,643.2$ Kg / d.

% reduction of total sludge = $\frac{(14,902 - 8,643.2) \times 100}{14,902} = 42$.

Concentration of sludge after digestion (X_{out}) = $8,643.2 / 321.51 = 26.9$ Kg / m³.

Solids content after digestion wt % = $100 X_{out} / \text{density} = \frac{26.9 \times 100}{1,000 \times 1.03} = 2.5603$.

4.2.9 Tertiary Treatment (Disinfection).

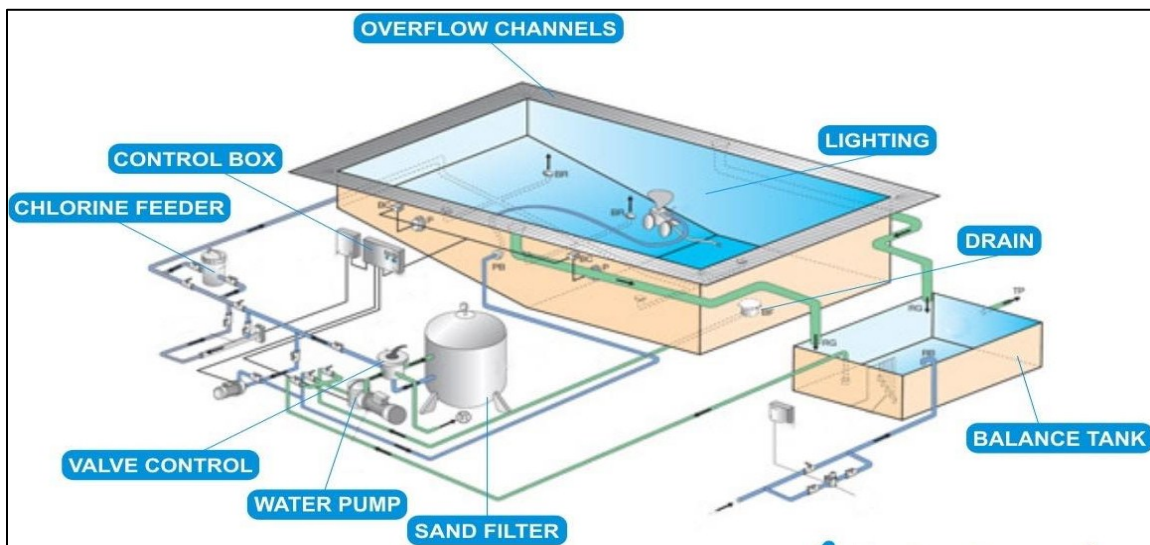


Figure (4.4): Chlorine Tank Design.

Step 1: Chlorine tank volume determination.

Type of bacteria = E_{coil}.

PH value = 8.

Water temperature = 20 °C.

% Kill of bacteria = 99.90.

C_t value table (16-2) l of bacteria = 99.90.

Concentration of microorganisms at time (t) = N_t .

Concentration of microorganisms at time (o) = N_o .

First order susceptibility coefficient = $K \text{ min}^{-1}$.

$$N_t / N_o \text{ for 1 log reduction} = \frac{(1 - 0.90) \times 100}{100} = 0.100.$$

1 log reduction is equivalent to % kill = 90.

2 log reduction is equivalent to % kill = 99.00.

$$N_t / N_o \text{ for 2 log reduction} = \frac{(1 - 0.99) \times 100}{100} = 0.0100.$$

3 log reduction is equivalent to % kill = 99.00.

$$N_t / N_o \text{ for 3 log reduction} = \frac{(1 - 0.999) \times 100}{100} = 0.00100.$$

4 log reduction is equivalent to % kill = 99.99.

$$N_t / N_o \text{ for 4 log reduction} = \frac{(1 - 0.9999) \times 100}{100} = 0.0001.$$

$a = 0.9847$, $b = 0.1758$, $c = 2.7519$, $d = -0.17467$.

Correlation:

$$C_t = aC^b + pH^c + T^d = 0.9847 \times 2.50^{0.1758} + 8^{2.7519} + 20^{-0.1467} = 307.44.$$

First order susceptibility coefficient = 0.20 hr^{-1} .

Chick's Law (1908) - most widely used $[dN/dt = -k \times N]$.

Where:

t = time between (20_30) min.

k = first order susceptibility coefficient.

N_t = concentration of living microorganism at time t .

N_0 = concentration of living microorganism at time 0.

Step 2: Assume plug flow.

In – out + generation = accumulation.

Step 3: Assume steady state.

$$Q dN + (-rN) dV = 0.$$

Where:

k = first order susceptibility coefficient min^{-1} .

Q , m^3 / sec .

$$-r = K \times N \times \theta.$$

$$\Theta = V / Q.$$

$$d\theta = dV / Q.$$

$$Q dN + (K \times V \times \theta) \times (d\theta \times Q) = 0.$$

$$dN + (K \times V \times \theta) d\theta = 0.$$

$$dN = -K \times N \times \theta \times d\theta.$$

$$dN / N = -K \times \theta \times d\theta.$$

$$\ln (N / N_0) = -K \times \theta^2 / 2.$$

$$\ln (0.001) = -0.345 \times (\theta^2 / 2).$$

$$\theta^2 = 2 \times \frac{-6.90776}{-0.07591} = 182.0 \text{ min}^2.$$

$$\Theta = 182^2 = 13.49 \text{ min.}$$

$$\text{Volume of chlorine tank} = 1.65205 \times 13.46 \times 60.$$

$$V = 1,337.25 \text{ m}^3.$$

$$\text{Constant of dilution (n)} = 0.82.$$

$$Ct^n (\text{mg} / \text{l-min}^n) \text{ correlation} = 307.44.$$

$$\text{Effective contact time} = 122.98 \text{ min.}$$

$$k (\text{pf}) = \text{constant for Plug flow.}$$

$$k (\text{pf}) = 0.20.$$

$$\text{For PF } \ln (C) = \ln (C_0) - k (\text{pf}).$$

$$\text{Exp } (- k (\text{pf}) \times t).$$

$$\text{Exp } (- 0.20 \times (122.98 / 60)) = 0.66370.$$

$$C_0 = C / [\text{Exp } (- k (\text{pf}) \times t)] = \frac{2.5}{0.66370} = 3.77.$$

$$\text{Treated water flow rate } (Q_e) = \frac{3.77 \times 142,737.45}{1,000} = 142,737.45 \text{ m}^3 / \text{d.}$$

$$\text{Quantity of chlorine consumed in PF} = 533.66 \text{ kg} / \text{d.}$$

$$\text{Type of chlorination basin is C.MIX completely mixed.}$$

$$t (\text{CM}) = t (\text{PF}) / 0.105 = (122.98 / 0.105) = 1,171.21 = 19.52 \text{ min.}$$

$$C_0 = 3.77 \times (1 + 19.52 \times 0.2) = 18.47 \text{ mg} / \text{L.}$$

$$\text{Quantity of chlorine consumed in CM} = \frac{142,737.45 \times 18.47}{1,000} = 2,636.7 \text{ kg} / \text{d.}$$

It is better to use pf than CM.

4.2.10 Nitrification Denitrification Calculations.

Table (4.1): Showing the Nitrification Denitrification Calculations.

Nitrogen group in wastewater	Raw municipal influent (mg/L)	Primary effluent (mg/L)	Secondary effluent no nitrification (mg/L)	Secondary effluent nitrified (mg/L)
NH ₄ ⁺ N	15	16	12	1
SKN	18	18	13	1.2
TKN	30	25	15	2.2
NO ₂ - N	0	0	0	0.1
NO ₃ - N	0	0	0	17
Total N	30	25	15	19.3
Sol. Organic-N	3	2	1	0.2
Organic-N	15	9	3	1.2

Table (4.2): Showing the Nitrification Denitrification Calculations (Cont.).

Parameter	Value	Parameter	Value
Average Temp in aeration tank, °C	20.00	Growth rate μ_m @ 25 °C	0.65 day ⁻¹
Average effluent NH ₃ as N, mg/L	1.00	Decay rate K _d @ 25 °C	0.05 day ⁻¹
Average DO conc in aerat, tank mg/L	3.00	Temp. correct function Φ	1.055
		Half saturation constant K _N	1.00 mg/L

Step 1: Calculating actual mean residence time (MCRT).

Aerobic volume = 23,179.94 m³.

$$\text{Anoxic vol.} = \frac{23,179.94}{3} = 7,726.65 \text{ m}^3.$$

$$\text{Aerobic zone MLSS} = 3,750 \text{ mg / L.}$$

$$\text{Sludge } Q_w = 579.50 \text{ m}^3 / \text{d.}$$

$$\text{TSS in waste sludge} = 15,000 \text{ mg / L.}$$

$$\text{SC effluent flow } Q_e = 142,737.40 \text{ m}^3 / \text{d.}$$

$$\text{SC effluent TSS} = 13 \text{ mg / L.}$$

$$\text{Volatile solids in WAS} = 0.8 \times 15,000 = 12,000 \text{ mg / l.}$$

$$\text{Mass of solids in (M-ANOX) zone} = \frac{7,726.65 \times 3,750}{100} = 28,974.93 \text{ Kg.}$$

$$\text{Mass of solids in (M-AER) zone} = \frac{23,179.94 \times 3,750}{1,000} = 86,924.78 \text{ kg.}$$

$$\text{Mass of solids rem (MW)} = Q_w (\text{TSS w}) \text{ Kg / d} = \frac{579.50 \times 15,000}{1,000} = 8,692.50.$$

$$\text{MCRT} = \frac{(M_{\text{ANOX}} + M_{\text{AER}})}{(M_w + M_e)} \quad (11)$$

$$\text{Mass of solids rem (ME)} = Q_e (\text{TSS eff}) = \frac{142,737.40 \times 13}{1,000} = 1,855.59 \text{ Kg / d.}$$

MCRT required for nitrification, days.

$$\text{MCRT} = \frac{28,974.92 + 8,692.78}{8,692.50 + 1,855.59} = 10.99.$$

$$\text{MCRTAER} = \frac{86,924.78 \times 10.99}{86,924.78 + 28,974.93} = 8.24.$$

$$\text{MCRTANOX} = 10.99 - 8.24 = 2.75.$$

Step 2: Nitrogen balance in nitrifying and de-nitrifying plant.

Table (4.3): Nitrification Denitrification Calculations.

Primary Effluent Flow, m³/d	138974.01	138974.01
<i>Parameter</i>	<i>mg/L</i>	<i>Kg/d</i>
Primary effluent TKN	25	3474.35
Primary effluent Nox-N	0	0.00
Nitrogen in waste activated sludge	6.7	931.13
Secondary effluent NH ₄ +N	1	138.97
Secondary effluent soluble Organoc-N	1	138.97
Nitrogen in secondary effluent SS, mg/L	1.2	166.77
Secondary effluent Nox-N, mg/L	6	833.84
Nitrogen Entering Activated Sludge		
Primary effluent TKN	25	3474.35
Primary effluent NO _x -N	0	0.00
<i>TOTAL</i>	<i>25</i>	<i>3474.35</i>
Nitrogen Leaving Activated Sludge		
N - in WAS	6.7	931.13
N-denitrified	X	X
Secondary effluent NH ₄ +N	1	138.97
Secondary effluent soluble Organoc-N	1	138.97
Secondary effluent N in TSS	1.2	166.77
Secondary effluent Nox-N	6	833.84
<i>TOTAL</i>	<i>15.9</i>	<i>2209.69</i>
N-denitrified Calculated by Difference (X)	<i>9.1</i>	<i>1264.66</i>

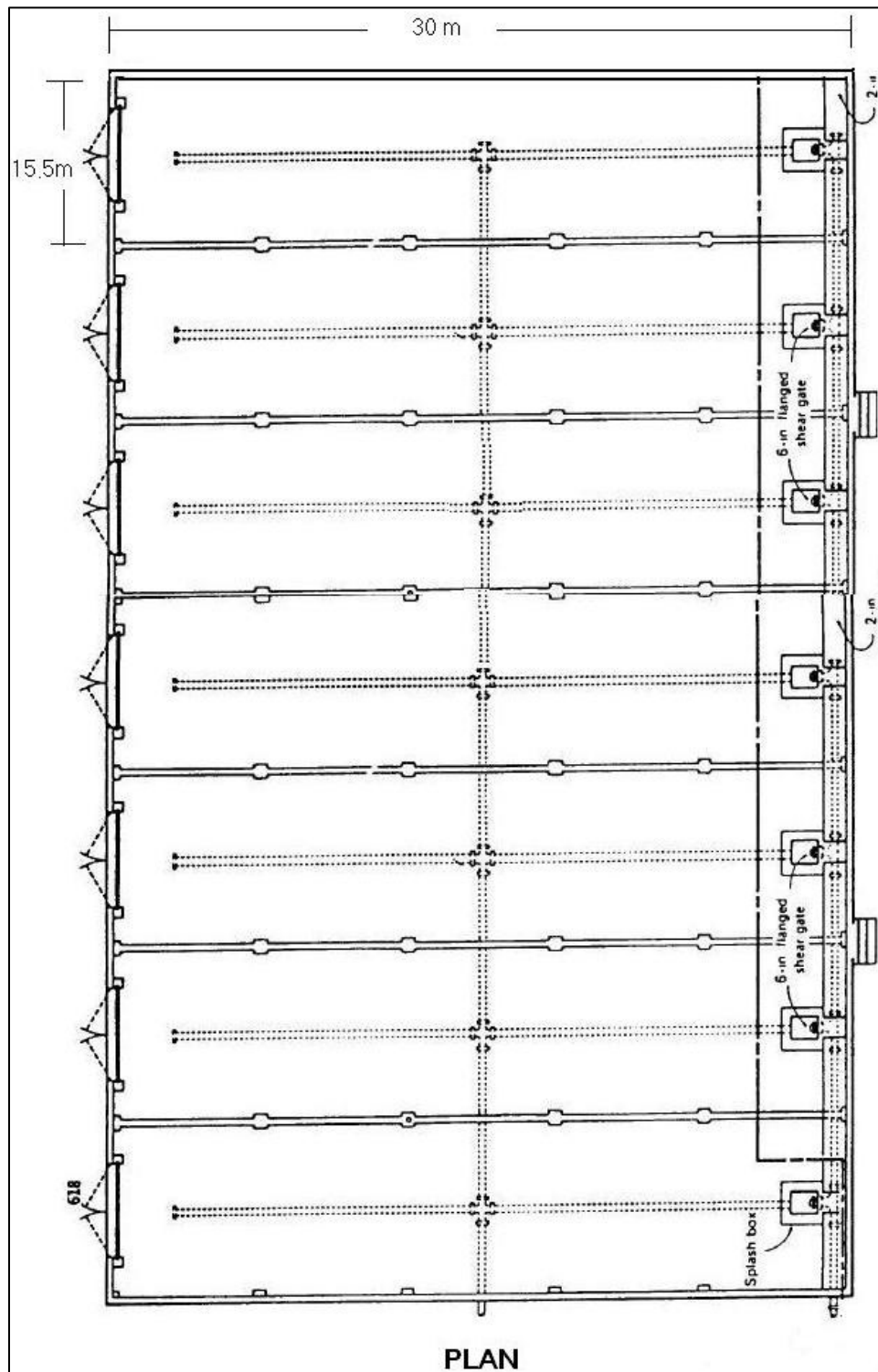


Figure (4.5): Design Details of Drying Beds.

4.3 Discussion and Conclusion:

- Notes that all the result was typical range, so all result satisfactory.
- The design of each stage of the treatment affected by the number of population. As the population increases, the results of each treatment process increase. We need to increase the design.
- Primary and secondary treatment removes most of the BOD and suspended solids, but this treatment has not been sufficient. An additional phase of treatment has added to remove organic matter, suspended solids, nutrients and toxic substances this phase is tertiary treatment.
- Dealing with Design's assumptions should carefully considered for each stages of the (WWTP).
- Biological treatment cannot completely remove nitrogen so a unit has been so a nitrogen-processing unit has added.
- Food to microorganism ratio (F/M) equal to (0.347) which is high ratio so that means the excess of air must be exist to remove BOD.
- The results up there, shown that the plant has good performance, which mean the efficiency of the plant based on soluble BOD equal to 95 %.

4.4 Recommendations:

- Notes that all the result was typical range, so all result satisfactory, this indicates that the project is very well and we can apply it.
- Provide conditions for bacterial growth such as presence of phosphorus, nitrogen and oxygen, as well as providing a suitable climate in terms of temperature and humidity and PH.
- Add detachment unit to remove oil and grease.

Reference.

- 1) Syed R. Qasim "Waste water treatment plant Planning, Design, and Operation ", (1985).
- 2) Sayed David A. Cornwell "Introduction to environmental engineering ", 1991. Benefield, L.D., And C.W. Randall, "Biological Process Design Of Waste Water Treatment", Prentice - Hall, Englewood Cliffs, N.J. 1980.
- 3) Donald w. Sundstron and Herbert E. Klei "Wastewater treatment "1985
- 4) Metcalf & Eddy., Tchobanoglous, G., Burton, F. L. 1. & Stensel, H. D. (2003). *Wastewater Engineering: Treatment and Reuse* (4th Ed.). Boston: McGraw-Hill.
- 5) Water Environment Federation – WEF – (1998). *Design of Municipal Wastewater Treatment Plants*, 4th Edition, and Manual of practice No. 8: Alexandria, VA, U.S.A.; American Society of Civil Engineers, Manual on Engineering Practice No. 76: New York, U.S.A.
- 6) Warren Viessman and Mark Hammer "water Supply and Pollution Control" Fifth edition (1993), Chapter 12, pages (561- 405),
- 7) Ronald L. Dorste "Theory and Practice of water and wastewater treatment" December (1997) John Wiley, Chapter 13.
- 8) Shun Dar Lin, and C. C. Lee "Water and Wastewater Calculation Manual "Second edition (2007), McGraw - Hill, Chapter 6.
- 9) Kalogo, Y, and Monteith, H.P., Water Environment Research Foundation – WERF – (2008). *State of Science Report: Energy and Resource Recovery from Sludge*. Jointly funded by WERF, UK Water Industry Research Limited, STOWA and sponsored by the Global Water Research Coalition (GWRC). Alexandria, VA, U.S.A.
- 10) Parry, D.L., Water Environment Research Foundation – WERF – (2014). *Co-Digestion of Organic Waste Products with Wastewater Solids – Final Report with Economic Model*. Alexandria, VA, U.S.A.
- 11) Hofkey, E.H. (1981), *Small Community Water Supplies, Technical Paper No 18*. The Hague, International Reference Centre.
- 12) Kalvin B. and Isley, R. (1984), *Developing and Using Audio – Visual and Water Supply Sanitation Programs*, Washington DC, USAID.

- 13) “Facultative Lagoons Treat Wastewater Naturally”, Small community wastewater issues to the public, Pipeline Journal spring 1997, Vol (8), No (2).
- 14) Allison Costa et.al. October 2015, USEPA “ Anaerobic digestion components and it’s applications” Land application and pollution control division, Cincinnati Ohio 45208
- 15) Margerum, D.W., Schurter, L.M., Hobson, J. & Moore E.E. 1994. Water chlorination chemistry: nonmetal redox kinetics of chloramines and nitrite ion. Environ. Sci. Tech. 28(2): 331-337
- 16) Lim Fang Yee¹, Md. Pauzi Abdullah^{1*}, Sadia Ata¹, Abbas Abdullah, Basar Ishak, and Khairul Nidzham “chlorination and chloroamine formation ”, The Malaysian Journal of Analytical Sciences, Vol (12), No 3 (2008): pages (528 – 535), School of Chemical Sciences and Food Technology, University Kebangsaan Malaysia,
- 17) Ayotte, R.C. & Gray, Jr. E.T. 1986. Chlorination of the peptide nitrogen. In water chlorination, chemistry, Environmental impact and health effect. (Vol. 5). Eds R.L. Jolley, R.J. Bull, W.P. Davis,
- 18) Roberts, Jr. M.H., Jacobs, V.A. Katz S. Lewis Publishers, Inc. Michigan. pg. 797-806. 9. Tachikawa, M., Takashi, A., Masakatsu, T. & Sawamura, R. 2005. Occurrence and production of chloramines in the chlorination of creatinine in aqueous solution. Water research. 39: 371-379. 10. APHA. 1998.
- 19) Bengtson, H.H., “Activated Sludge Wastewater Treatment Calculations with Excel Spreadsheets:” an online article at www.engineeringexcelspreadsheets.com
- 20) A source of low cost easy to use Excel Spreadsheets for Activated Sludge design and operational calculations at: www.engineeringexceltemplates.com

Appendices

Design Criteria

Appendix (A)

Table (A.1): Typical Contaminants Found in Untreated Wastewater.

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 common in terms of BOD (biochemical oxygen demand) and COD (chemical oxygen demand). 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 aquatic environment, 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.
Priority pollutants	Organic and inorganic compounds selected on the basis of their unknown or suspected carcinogenicity, mutagenicity, or high acute toxicity. The presence of these compounds in wastewater must be minimized for public health reasons and to protect the biological treatment processes.
Refractory organics	These organics tend to resist conventional methods of wastewater treatment. Typical examples include surfactants, phenols, and agricultural pesticides. Some of these may be toxic to the biological treatment processes.
Heavy metals	Heavy metals are usually added to wastewater from commercial and industrial activities and may have to be removed if the wastewater is discharged to a stream used as a potable water source. The presence of heavy metals may also impact the recycling of biosolids (stabilized waste sludge) on farmland.
Dissolved inorganics	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 discharged to a stream used as a potable water source.

Table (A.2): Typical Composition of Untreated Domestic Wastewater.

Contaminants	Unit	Concentration		
		Weak	Medium	Strong
Solids, total (TS)	mg/L	350	720	1200
Dissolved, total (TDS)	mg/L	250	500	850
Fixed	mg/L	145	300	525
Volatile	mg/L	105	200	325
Suspended solids (SS)	mg/L	100	220	350
Fixed	mg/L	20	55	75
Volatile	mg/L	80	165	275
Settleable solids	ml/L	5	10	20
Biochemical oxygen demand, mg/L: 5-day, 20°C (BOD ₅ , 20°C)	mg/L	110	220	400
Total organic carbon (TOC)	mg/L	80	160	290
Chemical oxygen demand (COD)	mg/L	250	500	1000
Nitrogen (total as N)	mg/L	20	40	85
Organic	mg/L	8	15	35
Free ammonia	mg/L	12	25	50
Nitrites	mg/L	0	0	0
Nitrates	mg/L	0	0	0
Phosphorus (total as P)	mg/L	4	8	15
Organic	mg/L	1	3	5
Inorganic	mg/L	3	5	10
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 coliform	no/100 mL	10 ⁶ -10 ⁷	10 ⁷ -10 ⁸	10 ⁷ -10 ⁹
Volatile organic compounds (VOCs)	µg/L	<100	100-400	>400

Appendix (B)

Table (B.1): Classification and Spacing of Screening Devices used in Wastewater Treatment Plants WWTP [4].

Type of screening device	Spacing distance
Coarse screening	(50-150) mm
Medium screening	(20-50) mm
Fine screening	(5-20) mm

Table (B.2): Typical Screening Design Factors used in Wastewater Treatment Plants WWTP [4].

Parameter	Value
Slope from horizontal (degrees)	(45-70)°
Clear spacing between bars	(10-40) mm
Velocity through rack	(0.3 – 1.0) m/sec
Allowable head loss, clogged screen	150 mm
Maximum head loss, clogged screen	80 mm



Figure (B.1): Mechanical Bar Screens.

Appendix (C)

Table (C.1): Typical Design Ranges for Bar Screens [4].

Parameter	Cleaning Method	
	Manual	Mechanical
Bar size width	5 – 15 mm	5 – 15 mm
Bar size depth	25 – 40 mm	25 – 40 mm
Clean spacing between bars	35 – 50 mm	6 – 75 mm
Slope from veridical	30 – 45 °	0 - 30°

Appendix (D)

Table (D.1): Design Factors and Typical Design Criteria Values
for Aerated Grit Chambers [4].

Parameter	Value
Grit chamber depth	(2 – 5) m
Grit chamber Length	(7.5 – 20) m
Width/depth ratio	(1:1) - (5:1)
Length/depth ratio	(2.5:1) - (5:1)
Transfer velocity at the surface	(0.6 – 0.8) m/sec
Detention time at maximum flow	(2 – 5) min
Air supply	(4.6 – 12.4) ℓ/ (sec -m of tank length)



Figure (D.1): Grit Chamber.

Appendix (E)

Table (E.1): Design Criteria of Primary Sedimentation Tanks [4].

Parameter	Range	Typical value
Detention time (hr)	1.5 – 2.5	2
Over flow rate (m/d) at Average flow	30 -50	40
Over flow rate (m/d) at Peak flow	80 - 120	90
Weir loading rate $\text{m}^3 /(\text{m}^2 - \text{day})$	120 - 450	200
BOD removal (%)	20 - 40	35
Total Suspended Solids removal (%)	30 - 70	65
Sludge acc. (Kg wet solids/ m^3 flow)	2 - 4	2.5
<i>Dimensions of a rectangular sedimentation tank (m)</i>		
Depth (m)	2 - 6	3.5
Length (m)	15 - 100	30
Width (m)	3 – 30	10
Sludge scraper speed (m/min)	0.5 – 1.5	1.0
<i>Dimensions of a circular sedimentation tank (m)</i>		
Depth (m)	3 - 5	4.5
Diameter (m)	3 - 60	30
Bottom slope (min/m)	60 - 160	80
Sludge scraper speed (r.p.m)	0.02 – 0.05	0.03

Appendix (F)

Table (F.1): Typical Design Information for Secondary Clarifiers for
Activated Sludge Wastewater Treatment [4].

Type of Treatment	Overflow rate		Solids Loading		Depth (m)
	$(m^3/(m^2\text{-day}))$		$(Kg/(m^2\text{-hr}))$		
	Average	Peak	Average	Peak	
Settling after Air A. Sludge	16 - 28	40 - 64	4 - 6	8	2.5 – 6
Settling biological Nutrient removal	16 - 28	40 - 64	5 - 8	9	2.5 – 6
Settling after O ₂ A. Sludge	16 - 28	40 - 64	5 - 7	9	2.5 – 6
Settling after extend aeration	8 - 16	24 - 32	1 - 5	7	2.5 – 6

Appendix (G)

Table (G.1): Design Criteria Required for Sludge Thickening Parameters [4].

<i>Parameters</i>	Type of Sludge				
	<i>Primary sludge</i>	<i>Active Sludge (low & half load)</i>	<i>Mixed Sludge (1st and 2nd)</i>	<i>Extended aeration Sludge.</i>	<i>Aerobic Digestion Sludge</i>
Solids load (kg/m ² -day)	<130	<35	<70	<35	<35
HLR (hours)	<1.4	<0.45	<0.9	<0.45	<0.45
Sludge HRT (hours)		>24	>24	>24	>24
Thickened sludge conc. %	8 – 10	2 – 3	4 - 7	25 - 35	2 -35

Appendix (H)

Table (H.1): Criteria and Design Procedure for Anaerobic Digester.

Design Criteria for Anaerobic Digester Design

1. Select anaerobic sludge digestion for stabilization of organic solids.
2. Provide two completely-mixed, high-rate anaerobic heated digesters with digestion temperature of 35°C.
3. The design flow to the sludge digester shall be equal to thickened sludge under the daily design flow condition.
4. Total volatile solids loading to the digester shall not exceed 3.6 kg/m³·day under extreme high loading condition.
5. The solids retention time at extreme high-flow condition shall not be less than 10 days.
6. The digester mixing shall be achieved by internal gas mixing.
7. The solids content in digested sludge is 5% and S.P. is 1.03.
8. The TSS content in the supernatant is 4,000 mg/L.
9. The ratio TVS/TS = 0.71, Y = 0.05 g VSS produced/g BOD₅ utilized, E = 0.8, and k_d = 0.03 1/day.

Table (H.2): Criteria and Design Procedure for Anaerobic Digester (continue).

Design Criteria for Anaerobic Digester Design - continued

10. The digester heating shall be achieved by recirculation of sludge through external heat exchanger. The sludge recirculation system shall also be designed to provide digester mixing.
11. Provide floating digester cover for gas collection.
12. The heat loss from the digester cover, side walls, and floor shall be calculated using the standard heat transfer coefficients for the digester construction material.
13. Provide gas-fired hot water boiler for external heat exchanger.
14. Explosion prevention devices shall be provided to minimize the possibility of an explosive mixture being developed inside the floating covers. Proper flame traps shall be provided to assure protection against the passage of flame into the digester, gas storage sphere, and supply lines.
15. The digester design shall include supernatant withdrawal system, sight glass, sampler, manhole, etc.
16. Arrangement shall be provided to break the scum that may form on the sludge surface.

Appendix (I)

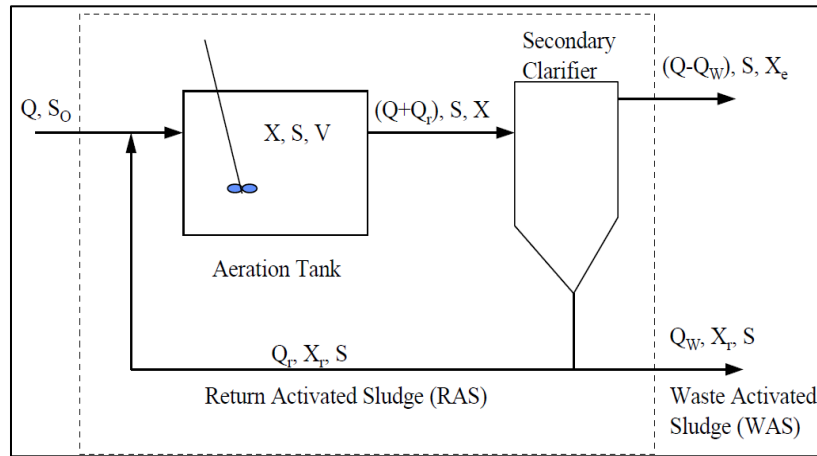


Figure (I.1): Completely Mixed Activated Sludge Material Balance and Design Equations.

Mass Balance:

Biomass:

$$QX_o + V \left(\frac{\mu_{\max} SX}{K_s + S} - k_d X \right) = (Q - Q_w) X_e + Q_w X_w$$

Substrate:

$$QS_o - V \left(\frac{\mu_{\max} SX}{Y(K_s + S)} \right) = (Q - Q_w) S_e + Q_w S_w$$

where Q, Q_w = influent flow and waste flow, respectively, m^3/d

V = volume of aeration basin, m^3

$\mu_{\max}$ = maximum specific growth rate coefficient, h^{-1}

K_s = half saturation coefficient, mg/L

k_d = decay coefficient, h^{-1}

X_o, X, X_e, X_w = biomass in influent, bioreactor, effluent, and waste, mg/L as MLVSS

S = soluble substrate concentration in bioreactor, mg/L as BOD or COD

S_o = influent substrate concentration, mg/L as BOD or COD

Y = biomass yield, mg biomass formed/ mg substrate utilized (mg VSS/ mg BOD)

Assumptions:

1. Influent and effluent _____ concentration is negligible
2. Aeration basin is a _____ CSTR, $S = S_w = S_e$
3. All reactions occur in _____ basin

Then:

$$\frac{\mu_{\max} S}{K_S + S} = \frac{Q_w X_w}{VX} + k_d \quad ; \quad \frac{\mu_{\max} S}{K_S + S} = \frac{QY}{VX}(S_o - S)$$

Observe:

$$\frac{Q}{V} = \frac{1}{\theta} \quad ; \quad \frac{Q_w X_w}{VX} = \frac{1}{\theta_c}$$

Where θ = the hydraulic retention time, HRT, and θ_c = the solids residence time, SRT. This results in the following design equations:

$$S = \frac{K_S (1 + k_d \theta_c)}{\theta_c (\mu_{\max} - k_d) - 1} \quad ; \quad \theta_c = \frac{K_S + S}{S (\mu_{\max} - k_d) - K_S k_d} \quad ; \quad X = \frac{\theta_c Y (S_o - S)}{\theta (1 + k_d \theta_c)}$$

The minimum soluble BOD concentration that can be achieved as $\theta_c \rightarrow \infty$:

$$S_{\min} = \frac{K_S k_d}{\mu_{\max} - k_d}$$

The minimum θ_c achievable as $\mu \rightarrow \mu_{\max}$:

$$\theta_{c_{\min}} = \frac{K_S + S_o}{S_o (\mu_{\max} - k_d) - K_S k_d}$$

Steps for Activated Sludge Design

1. Establish effluent soluble BOD₅ allowable to meet BOD₅ and SS effluent limits.
2. Determine what θ_c is required to meet the effluent soluble BOD₅ allowable.
3. Solve for the mixed liquor volatile suspended solids, MLVSS, concentration given particular hydraulic residence time, θ . Or solve for θ given a particular MLVSS.
4. Calculate the return activated sludge (RAS) flow, Q_r , and concentration, X_r' .

$$X_r' Q_r = X(Q_r + Q) \quad ; \quad X_r' = 10^6 / \text{SVI}$$

where $X' = \text{MLSS, mg/L}$ (X' typically is approximately $1.2 \cdot X$)

$X_r' = \text{RAS concentration, mg/L}$

$Q_r = \text{RAS flow rate, m}^3/\text{s}$

Table (I.1): Typical Contaminants Found in Untreated Wastewater.

AST Type	SRT [§] (days)	F/M (lb BOD/lb MLVSS day)	Vol. Loading (lb BOD/1000 ft ³ day)	Reactor * MLSS (mg/L)	Reactor * HRT (hrs)	RAS Flow (% of influent)
High-Purity Oxygen	1 - 4	0.5 - 1.0	80 - 200	2000 - 5000	1 - 3	25 - 50
Contact Stabilization [^]	5 - 10	0.2 - 0.6	60 - 75	1000 - 3000	0.5 - 1	25 - 150
Conv. Complete Mix	3 - 15	0.2 - 0.6	20 - 100	1500 - 6000	3 - 5	25 - 100
Conv. Plug Flow	3 - 15	0.2 - 0.4	20 - 40	1000 - 3000	4 - 8	25 - 75
Step Feed	3 - 15	0.2 - 0.4	40 - 60	1500 - 4000	3 - 5	25 - 75
Seq. Batch Reactor	10 - 30	0.04 - 0.10	5 - 15	2000 - 5000	15 - 40	NA [#]
Oxidation Ditch	15 - 30	0.04 - 0.10	5 - 15	3000 - 5000	15 - 35	75 - 150
Extended Aeration	20 - 40	0.04 - 0.10	5 - 15	2000 - 6000	20 - 35	50 - 150

[^] MLSS and HRT values shown are for the contact tank. For stabilization tank, MLSS = 4000 - 10,000 mg/L and HRT = 2 - 6 hours.

[§] Based on solids in aeration tank only

* Reactor = Aeration tank

[#] NA = not applicable

Table based on Metcalf & Eddy (2003) and Water Pollution Control Federation (1987) for treatment of municipal wastewater

Appendix (J)

Table (J.1): Nitrification Denitrification Calculations.

Primary Effluent Flow, m³/d	138974.01	138974.01
<i>Parameter</i>	<i>mg/L</i>	<i>Kg/d</i>
Primary effluent TKN	25	3474.35
Primary effluent Nox-N	0	0.00
Nitrogen in waste activated sludge	6.7	931.13
Secondary effluent NH ₄ +N	1	138.97
Secondary effluent soluble Organoc-N	1	138.97
Nitrogen in secondary effluent SS, mg/L	1.2	166.77
Secondary effluent Nox-N, mg/L	6	833.84
Nitrogen Entering Activated Sludge		
Primary effluent TKN	25	3474.35
Primary effluent NOx-N	0	0.00
<i>TOTAL</i>	<i>25</i>	<i>3474.35</i>
Nitrogen Leaving Activated Sludge		
N - in WAS	6.7	931.13
N-denitrified	X	X
Secondary effluent NH ₄ +N	1	138.97
Secondary effluent soluble Organoc-N	1	138.97
Secondary effluent N in TSS	1.2	166.77
Secondary effluent Nox-N	6	833.84
<i>TOTAL</i>	<i>15.9</i>	<i>2209.69</i>
N-denitrified Calculated by Difference (X)	9.1	1264.66