DESIGN AND DEVELOPMENT OF DOMESTIC WASTE WATER TREATMENT PLANT FOR IRRIGATION PURPOSE

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MATRIC No. 2005/21667EA

DEPARTMENT OF AGRICULTURAL AND BIORESOURCES ENGINEERING

FEDERAL UNIVERSITY OF TECNOLOGY, MINNA

DECEMBER, 2010

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BEING A FINAL YEAR PROJECT SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF BACHELOR OF ENGINEERING (B.ENG) DEGREE IN AGRICULTURAL AND BIORESOURCES ENGINEERING

FEDERAL UNIVERSITY OF TECNOLOGY, MINNA

DECEMBER, 2010

DECLARATION

I hereby declare that this project is a record of a research work that was undertaken and written by me. It has not been presented before for any degree or diploma or certificate at any University or institution. Information derived from personal communications, published and unpublished works of others were duly reference in the texts.

SALAU, MOHAMMED

8/12/10

DATE

CERTIFICATION

This is to certify that the project entitled "Design and Development of Domestic Wastewater Treatment Plant for Irrigation purposes" by Salau, Mohammed meets the regulations governing the award of the degree of Bachelor of Engineering (B. ENG.) of the Federal University of Technology, Minna, and it is approved for its contribution to scientific knowledge and literary presentation.

Engr. Mrs. H.I. Mustapha Supervisor	13/12/10 Date
Engr. Dr. A.A. Balami Head of Department	15/12/10 Date
External Examiner	15/12/2010 Date

DEDICATION

This research work is dedicated to Allah (SWT) and his prophet Muhammad (SAW)

ACKNOWLEDGEMENT

All the glory to ALLAH (S.W.T) Who made it possible for the completion of this research work. I am grateful for the strength and wisdom He gave me to accomplish this task.

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ABSTRACT

The domestic waste water treatment plant was developed, designed and constructed with the intention of controlling the pathogenic and chemical load of waste water in the course of using it for irrigation. On evaluating the plant, it brought down the Biochemical Oxygen Demand level of the waste water from 100mg/l to 0.4mg/l far below the 2.0 mg/l set by the WHO standard for irrigation water quality. It also brought down the pH level from 9.16 to 8.4, which is within the acceptable range of WHO of 6.5 to 8.5. The dissolved oxygen in influent waste water into the treatment plant was 1.29mg/l, it was brought down to 0.70mg/l, however, this is not supposed to decrease, it implies that the detention time for aeration is not enough, the dissolved oxygen content did not meet WHO required range of 2.0 to 7.5 mg/l. The domestic waste water treatment plant was effective in controlling the pathogenic and chemical load of the waste water.

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CHAPTER ONE

1.0 INTRODUCTION

1.1 Background

The supply of clean water is an essential requirement for the establishment and maintenance of healthy community. It act not only as source of potable water, but also provides valuable food supplements through supporting the growth of aquatic life and also by it usage for irrigation in agriculture. Water which has been utilized and discharge from domestic dwellings, institutions and commercial establishments are known as domestic waste water. It contains a large number of potentially harmful compounds (WHO, 2007).

The use of waste water that contain bathwater, water from the kitchen and laundry water around the environs of Federal University Of Technology Minna, Niger State, Nigeria can no longer be ignored, much waste water is released into the river flowing behind the school campus and this has become a major irrigation water source in the dry season. Thus, as this trend cannot be changed, it becomes necessary that a thorough analysis of this waste water sample be undertaken to determine its degree of usefulness and adverse effects it has on human health and the environment. This should also lead to the evolvement of a process by which if applied, the adverse effects of waste water use for irrigation will be brought to the barest minimum possible.

In many arid and semi-arid countries water is becoming an increasingly scare resource and planners are forced to consider any sources of water which might be used economically and effectively to promote further development. At the same time, with population expanding at high rate the need for increased food production is apparent. The potential for irrigation to raise both agricultural productivity and the living standards of the rural poor has long been recognized.

Irrigated agriculture land occupies approximately 17% of world's total arable land comprises about 34% of the world total. This potential is even more pronounced in arid areas, such as the mid east region, where only 30% of cultivated land is irrigated but it produce about 75% of the total agricultural production. In this same region, more than 50% of the food requirements are imported and the rate of increase in demand for food exceeds the rate of increase in agricultural production (Salvato, 1992).

Whenever good quality water is scarce, water of marginal quality will have to be considered for use in agriculture. Although there is no universal definition of marginal quality water, for all practical purposes it can be defined as water that possesses certain characteristic which have the potential to cause problems when it is used for an intended purpose.

For example, brackish water is marginal quality water for agriculture use because of its high dissolved salt content, and municipal wastewater is marginal quality water because of the associated health hazards.

From the viewpoint of irrigation, use of marginal quality water requires more complex management practice and more stringent monitoring procedure than when good quality water is used. This project deals with the agricultural use of waste water which is primarily domestic sewage water.

Expansion of urban populations and increased coverage of domestic water supply and sewerage gives rise to greater quantities of municipal waste water. With the emphasis on environmental health and pollution issues, there is an increasing awareness of the need to dispose of these waste water in agriculture could be an important consideration when its disposal is being planned in arid and semi-arid regions. However, it should be realized that the quantity of waste water available in most countries will account for only a small fraction of the total irrigation water requirements. Nevertheless, waste water use will result in the conservation of higher quality water and its use for purpose other than irrigation.

As the marginal cost of alternative supplies of good quality water will usually be higher in water-short areas, it makes good sense to incorporate agricultural reuse into water resource and land use planning.

Properly planned use of municipal waste water alleviates surface water pollution problems contained in sewage to grow crops. The available of this additional water near population center will increase the choice of crop which farmers can grow. The nitrogen and phosphorus content of sewage might reduce or eliminate the requirements for commercial fertilizer. It is advantageous to consider effluent reuse at the same time as waste water collection treatment and disposal are planned so that sewerage system design can be optimized in terms of effluent transport and treatment methods. The cost of transmission of effluent from inappropriately sited sewage treatment plants to distant agricultural land is usually prohibitive. Additionally, swage treatment techniques for effluent discharge to surface waters may not always be appropriate for agriculture use of effluent.

Many countries of the world have included waste water reuse as an important dimension in water resources planning. In the more arid areas of Australia and USA, waste water is used in agriculture, releasing high quality water supplies for potable use. Some countries, for example, the kingdom of Jordan and the kingdom of Saudi Arabia, have a national policy to reuse all treated waste water effluents and have already made considerable progress towards this end. In China, sewage use in agriculture had developed rapidly since 1958 and now over 1.33 million hectares are irrigated with sewage effluent. (FAO, 1992).

In Northern Nigeria during the dry seasons, waste water effluent is used for agricultural reuse.

Moreover, the water in this case is untreated and this poses a lot of hazards for the health of the final consumers of agricultural products. Thus, the purpose of this project is to evolve and design a simple waste water treatment plant which can be accessible and affordable by local farmers, since most countries of sub-Saharan Africa and the third world cannot afford to treat all waste water produced in their countries. Less than 1% of waste water is treated in Nigeria (Aberuagba, 2001).

1.2 Aims

To established physio-chemical and bacteriological design parameters for the treatment of domestic waste water into a waste water treatment plant.

1.2.1 Objectives

- To construct a waste water treatment plant based on the design specification.
- To carry out a performance evaluation of the waste water treatment plant.
- To determine efficiency of the waste water treatment plant.

1.3 Scope and Limitation of the Study

The scope of this work covers the design, construction and evaluation of a treatment plant for domestic waste water only.

1.4 Project Justification

This project is of utmost importance to the society and environment because it seeks to:

- Eliminate harmful effects of the use of waste water for irrigation processes.
- Stop the contamination of groundwater by untreated waste water.
- Stop the creation of habitats for disease vectors.
- Improve soil conditions by preventing the build up of chemical pollutants in soil.
- Stop the contamination of surface water by untreated waste water.

CHAPTER TWO

2.0 LITERATURE REVIEW

Water treatment is a process of removing undesirable chemicals, materials, and biological contaminants from raw water. The goal is to produce water fit for a specific purpose. Most water is purified for human consumption (Drinking water) but water purification may also be designed for a variety of other purposes, including meeting the requirements of medical, pharmacology, chemical and agricultural purposes. In general the methods used include physical process such as filtration and sedimentation, biological processes such as slow sand filters or activated sludge, chemical process such as flocculation (WHO, 2005).

2.1 Screening

Screens is the very first operation carried out at a sewage treatment plant, and consists of passing the sewage through different types of screens, so as to trap and remove the floating matter, such as pieces of cloth, paper, wood, kitchen refuse etc. present in sewage. These floating materials, if not removed will chock the pipes or adversely affect the working of the sewage pumps. Thus the main idea of providing screens is to protect the pumps and other equipments from the possible damages due the floating matter of the sewage. It should be placed before the grit chambers. However if the quality of grit is not of much importance as in the case of landing filling etc. screens may even be placed after the grit chambers. They may sometimes be accommodated in the body of the grit chambers themselves (Massoud and Ahmad, 2005).

2.2 Primary Treatment

In the primary sedimentation stage, sewage flows through large tanks, commonly called "primary clarifiers" or "primary sedimentation tanks". The tanks are used to settle sludge while grease and oils rise to the surface and are skimmed off. Primary settling tanks are usually equipped with mechanically driven scrapers that continually drive the collected sludge towards a hopper in the base of the tank where it is pumped to sludge treatment facilities. Grease and oil from the floating material can sometimes be recovered for saponification. The dimensions of the tank should be designed to effect removal of a high percentage of the floatables and sludge. A typical sedimentation tank may remove from 60% to 65% of suspended solids, and from 30% to 35% of BOD from the sewage (Khopkar, 2004).

2.2.1 Odour Control

Odours emitted by sewage treatment are typically an indication of an anaerobic or "septic" condition. Early stages of processing will tend to produce smelly gases, with hydrogen sulfide being most common in generating complaints. Large process plants in urban areas will often treat the odours with carbon reactors, a contact media with bio-slimes, small doses of chlorine, or circulating fluids to biologically capture and metabolize the obnoxious gases. Other methods of odour control exist, including addition of iron salts, hydrogen peroxide, calcium nitrate, etc. to manage hydrogen sulfide levels (n.d, 2010).

2.3 Secondary Treatment

As well as further separating the finer organic solids from the liquids, the secondary stage of treatment removes biological nutrients, in particular nitrogen, by way of the 'activated sludge' process.

2.3.1 Activated Sludge Reactors

These are tanks which receive liquid from the primary sedimentation tanks. Activated sludge is a biological process in which microorganisms or "bugs" convert the organic matter and other constituents in the wastewater into gases and cell tissues. Ammonia in the wastewater is converted into nitrate, its more stable form. This is achieved in a number of compartments in the tank, in both the presence and absence of oxygen. Large mechanical blower units pump vast quantities of air through a series of pipes and diffusers and deliver many tiny air bubbles to the bottom of the tank. This vigorous aeration forms particles of "activated sludge", which contain large numbers of active bacteria, together with ciliates, rotifers and protozoa. Micro-organisms grow rapidly under these conditions, consuming and growing on the organic material in the wastewater.

2.3.2 Clarifiers

After leaving the activated sludge reactor the liquid flows to large circular tanks known as clarifiers. These tanks allow the liquid to decant before gravitating to the inlet end of the first lake. The sludge, which settles to the floor of the clarifier is removed and returned to the inlet end of the activated sludge reactor where it mixes with the incoming effluent to continue the biological process. The activated sludge process usually takes between four and eight hours and the advantage of this method of treatment is that relatively little land area is required, and the quality of the final effluent is high. Some sludge has to be discarded from the activated sludge reactor on a daily basis. This 'wasted' activated sludge is very watery and is thickened in the dissolved air flotation (DAF) tank prior to it being pumped to the cell lysis plant and then to the sludge digesters (n.d, 2010).

2.4 Tertiary Treatment

Tertiary treatment typically involves disinfection to destroy any remaining harmful bacteria and viruses, to make the wastewater suitable for discharge to the marine environment.

2.5 Sedimentation

Waters exiting the flocculation basin may enter the sedimentation basin, also called a clarifier or settling basin. It is a large tank with slow flow, allowing floc to settle to the bottom. The sedimentation basin is best located close to the flocculation basin so the transit between does not permit settlement or floc break up. Sedimentation basins may be rectangular, where water flows from end to end, or circular where flow is from the centre outward. Sedimentation basin outflow is typically over a weir so only a thin top layer that furthest from the sediment exits. The amount of floc that settles out of the water is dependent on basin retention time and on basin depth. The retention time of the water must therefore be balanced against the cost of a larger basin. The minimum clarifier retention time is normally 4 hours. A deep basin will allow more floc to settle out than a shallow basin. This is because large particles settle faster than smaller ones, so large particles collide with and integrate smaller particles as they settle. In effect, large particles sweep vertically through the basin and clean out smaller particles on their way to the bottom .As particles settle to the bottom of the basin, a layer of sludge is formed on the floor of the tank. This layer of sludge must be removed and treated. The amount of sludge that is generated is significant, often 3 to 5 percent of the total volume of water that is treated. The cost of treating and disposing of the sludge can be a significant part of the operating cost of a water treatment plant. The tank may be equipped with mechanical cleaning devices that continually clean the bottom of the tank or the tank can be taken out of service when the bottom needs to be cleaned (Gilbert, 1998).

2.5.1 Sludge Disposal

Waste water treatment processes produce large volumes of sludge. The treatment and disposal of the sludge is one of the major problems in wastewater treatment and accounts for about 50% of the total cost. As the sludge will undergo decay and produce offensive odors it has to be disposed off in a proper way.

2.5.2 Disposal of Treated Sewage or Sludge Disposal

Sewage cannot be simply disposed off due to their microbiological and chemical characteristics. Only after proper treatments they can be discharged, that too in a proper way. Disposal of treated sewage into water bodies and lands and recycling of waste water have raised objections from the public. So the methods of sewage disposal should ensure certain percent safety to get public acceptance. The methods for the disposal of sludge and effluent are different from each other.

2.5.3 Disposal Into Sea

In treatment plants which are located near coastal areas disposal of sludge into sea can be practiced. If the site of disposal is deep water areas there will not be any environmental problems. However, agitations have been raised worldwide against the disposal into the sea.

2.5.4 Disposal By Land Filling

The increased costs of sludge disposal paved the way to develop new technologies which will reduce the amount of sludge produced in sewage treatment. Water treatment produces less sludge than the sewage treatment. Moreover sludge from water treatment has low organic content and hence can be disposed by landfill without any objection. Sludge from softening process contains high content of calcium and in this way desirable for agricultural use.

Sewage sludge can also be used as a landfill material thus becoming a useful way for land reclamation. For example, it can be dumped in old quarries. But there is a possibility of groundwater being contaminated.

2.5.5 Disposal By Incineration

Where transport cost for sludge disposal is high, for example if the disposal site is far away from the treatment plant, incineration is the best alternative. Incineration is also particularly useful in cases in which the sludge contains toxic chemicals making their disposal environmentally unacceptable through other routes. Rotary multiple hearth type incinerators are widely used for sludge incineration. Fluidized bed furnaces are also becoming popular. Dewatered sludge with about 30% solid content has sufficient calorific value to maintain self-sustaining combustion. However, most incinerators require fuel oil for efficient combustion. After incineration the resulting inert ash is easily disposed. Still the sludge disposal by incineration poses a potential environmental problem.

2.5.6 Centrifugation

This method has not been practiced much as it is less efficient in dewatering and is a continuous process. It employs a solid-bowl type centrifuge. Conditioned sludge is fed to the centre of bowl. Rapid rotation of the bowl throws the solid to the outer edge of the bowl. Scraper blades remove the solids deposited on the edge.

In all the above four methods the liquid portion separated from the sludge must be properly disposed. Depending upon the organic content, it may be either disposed directly or sent back to the treatment plant for further stabilization (n.d, 2010).

2.6 Aeration

Aeration Basin supply large amounts of air to the mixture of primary wastewater and helpful bacteria and the other microorganisms that consume the harmful organic matter. The growth of the helpful microorganisms is sped up by vigorous mixing of air (aeration) with the concentrated microorganisms (activated sludge) and the wastewater. Adequate oxygen is supplied to support the biological process at a very active level. The ratio of food (organic matter) to organisms to oxygen is continually monitored and adjusted to meet daily variations in the wastewater. It is used in the treatment of raw water (meat for human consumption) for purpose of releasing entrained gases, adding oxygen, reducing iron and manganese content, odor and generally improving the chemical and physical characteristics of the water. It is also used in the biological treatment of domestic wastewater for the purpose of reducing the BOD of the organic matter content in the water. This is done by converting organic matter to cell tissues. These cell tissues are then subsequently removed. The conversion is done using oxygen.

A Typical Surface-Aerated Basin (using motor-driven floating aerators)

Most biological oxidation processes for treating industrial wastewaters have in common the use of oxygen (or air) and microbial action. Surface-aerated basins achieve 80 to 90% removal of Biochemical Oxygen Demand with retention times of 1 to 10 days. The basins may range in depth from 1.5 to 5.0 meters and use motor-driven aerators floating on the surface of the wastewater. In an aerated basin system, the aerators provide two functions: they transfer air into the basins required by the biological oxidation reactions, and they provide the mixing required for dispersing the air and for contacting the reactants (that is, oxygen, wastewater and microbes). Typically, the floating surface aerators are rated to deliver the amount of air equivalent to 1.8 to 2.7 kg 0_2/kW·h. However, they do not provide as good mixing as is normally achieved in activated sludge systems and therefore aerated basins do not achieve the same performance level as activated sludge units. Biological oxidation processes are sensitive to temperature and, between 0 °C and 40 °C, the rate of biological reactions increase with temperature. Most surface aerated vessels operate at between 4 °C and 32 °C (Gilbert, 1998).

2.6.1 Filter Beds (Oxidizing Beds)

In older plants and those receiving variable loadings, trickling filter beds are used where the settled sewage liquor is spread onto the surface of a bed made up of coke (carbonized coal), limestone chips or specially fabricated plastic media. Such media must have large surface areas to support the biofilms that form. The liquor is typically distributed through perforated spray arms. The distributed liquor trickles through the bed and is collected in drains at the base. These drains also provide a source of air which percolates up through the bed, keeping it aerobic. Biological films of bacteria, protozoa and fungi form on the media's surfaces and eat or otherwise reduce the organic content. This biofilm is often grazed by insect larvae, snails, and worms which help maintain an optimal thickness. Overloading of beds increases the thickness of the film leading to clogging of the filter media and ponding on the surface.

2.6.2 Biological Aerated Filters

Biological Aerated (or Anoxic) Filter (BAF) or Biofilters combine filtration with biological carbon reduction, nitrification or gentrification. BAF usually includes a reactor filled with a filter media. The media is either in suspension or supported by a gravel layer at the foot of the filter. The dual purpose of this media is to support highly active biomass that is attached to it and to filter suspended solids. Carbon reduction and ammonia conversion occurs in aerobic mode and sometime achieved in a single reactor while nitrate conversion occurs in anoxic mode. BAF is operated either in up flow or down flow configuration depending on design specified by manufacturer (Kingston, 1998).

2.7 Filtration

After separating most floc, the water is filtered as the final step to remove remaining suspended particles and unsettled floc.

2.7.1 Rapid Sand Filters

The most common type of filter is a rapid sand filter. Water moves vertically through sand which often has a layer of activated carbon or anthracite coal above the sand. The top layer removes organic compounds, which contribute to taste and odour. The space between sand particles is larger than the smallest suspended particles, so simple filtration is not enough. Most particles pass through surface layers but are trapped in pore spaces or adhere to sand particles. Effective filtration extends into the depth of the filter. This property of the filter is key to its operation: if the top layer of sand were to block all the particles, the filter would quickly clog. To clean the filter, water is passed quickly upward through the filter, opposite the normal direction (called backflushing or backwashing) to remove embedded particles. Prior to this, compressed air may be blown up through the bottom of the filter to break up the compacted filter media to aid the backwashing process; this is known as air scouring. This contaminated water can be disposed of, along with the sludge from the sedimentation basin, or it can be recycled by mixing with the raw water entering the plant. Some water treatment plants employ pressure filters. These works on the same principle as rapid gravity filters, differing in that the filter medium is enclosed in a steel vessel and the water is forced through it under pressure.

2.7.2 Advantages:

- Filters out much smaller particles than paper and sand filters can.
- Filters out virtually all particles larger than their specified pore sizes.
- They are quite thin and so liquids flow through them fairly rapidly.
- They are reasonably strong and so can withstand pressure differences across them of typically 2-5 atmospheres.
- They can be cleaned (back flushed) and reused.

Membrane filters are widely used for filtering both drinking water and sewage (for reuse). For drinking water, membrane filters can remove virtually all particles larger than 0.2 um including giardia and cryptosporidium. Membrane filters are an effective form of tertiary treatment when it is desired to reuse the water for industry, for limited domestic purposes, or before discharging the water into a river that is used by towns further downstream. They are widely used in industry, particularly for beverage preparation (including bottled water). However no filtration can remove substances that are actually dissolved in the water such as phosphorus, nitrates and heavy metal ions (Hobbins, 2003).

2.7.3 Slow Sand Filters

Slow sand filters may be used where there is sufficient land and space as the water must be passed very slowly through the filters. These filters rely on biological treatment processes for their action rather than physical filtration. The filters are carefully constructed using graded layers of sand with the coarsest sand, along with some gravel, at the bottom and finest sand at the top. Drains at the base convey treated water away for disinfection. Filtration depends on the development of a thin biological layer, called the zoogleal layer or Schmutzdecke, on the surface of the filter. An effective slow sand filter may remain in service for many weeks or even months if the pre-treatment is well designed and produces water with a very low available nutrient level which physical methods of treatment rarely achieve. Very low nutrient levels allow water to be safely sent through distribution system with very low disinfectant levels thereby reducing consumer irritation over offensive levels of chlorine and chlorine by-products. Slow sand filters are not backwashed; they are maintained by having the top layer of sand scraped off when flow is eventually obstructed by biological growth. A specific 'large-scale' form of slow sand filter is the process of bank filtration, in which natural sediments in a riverbank are used to provide a first stage of contaminant filtration. While typically not sufficiently clean enough to be used directly for drinking water, the water gained from the associated extraction wells is much less problematic than river water taken directly from the major streams where bank filtration is often used (Gilbert,1998).

2.7.4 Lava Filters

Lava filters are similar to sand filters and may also only be used where there is sufficient land and space. Like sand filters, the filters rely on biological treatment processes for their action rather than physical filtration. Unlike slow sand filters however, they are constructed out of 2 layers of lava pebbles and a top layer of nutrient-free soil (only at the plant roots). On top, water-purifying plants (such as Iris pseudacorus and Sparganium erectum) are placed. Usually, around 1/4 of the dimension of lavastone is required to purify the water and just like slow sand filters, a series of herringbone drains are placed with lava filters these are placed at the bottom layer (Kozisek, 2004).

2.8 Removal of Ions and Other Dissolved Substances

Ultrafiltration membranes use polymer membranes with chemically formed microscopic pores that can be used to filter out dissolved substances avoiding the use of coagulants. The type of membrane media determines how much pressure is needed to drive the water through and what sizes of micro-organisms can be filtered out.

Ion exchange: Ion exchange systems use ion exchange resin- or zeolite-packed columns to replace unwanted ions. The most common case is water softening consisting of removal of Ca^{2+} and Mg^{2+} ions replacing them with benign (soap friendly) Na^+ or K^+ ions. Ion exchange resins are also used to remove toxic ions such as nitrate, nitrite, lead, mercury, arsenic and many others.

2.8.1 Electrodeionization

Water is passed between a positive electrode and a negative electrode. Ion exchange membranes allow only positive ions to migrate from the treated water toward the negative electrode and only negative ions toward the positive electrode. High purity deionized water is produced with a little worse degree of purification in comparison with ion exchange treatment. Complete removal of ions from water is regarded as electrodialysis. The water is often pre-treated with a reverse osmosis unit to remove non-ionic organic (Harland, 1994).

2.8.2 pH Adjustment

Distilled water has a pH of 7 (neither alkaline nor acidic) and sea water has an average pH of 8.3 (slightly alkaline). If the water is acidic (lower than 7), lime, soda ash, or sodium hydroxide is added to raise the pH. For somewhat acidic, alkaline waters (lower than 6.5), forced draft degasifiers are the cheapest way to raise the pH, as the process raises the pH by stripping dissolved carbon dioxide (carbonic acid) from the water. Lime is commonly used for pH adjustment for municipal water, or at the start of a treatment plant for process water, as it is cheap, but it also increases the ionic load by raising the water hardness. Making the water slightly alkaline ensures that coagulation and flocculation processes work effectively and also helps to minimize the risk of lead being dissolved from lead pipes and lead solder in pipe fittings. Acid (HCl or H_2SO_4) may be added to alkaline waters in some circumstances to lower the pH. Having alkaline water does not necessarily mean that lead or copper from the plumbing system will not be dissolved into the water but as a generality, water with a pH above 7 is much less likely to dissolve heavy metals than a water with a pH below 7 (Zagorodni, 2006).

2.8.3 Flocculation

Flocculation is a process which clarifies the water. Clarifying means removing any turbidity or colour so that the water is clear and colourless. Clarification is done by causing a precipitate to form in the water which can be removed using simple physical methods. Initially the precipitate forms as very small particles but as the water is gently stirred, these particles stick together to form bigger particles - this process is sometimes called flocculation. Many of the small particles that were originally present in the raw water adsorb onto the surface of these small precipitate particles and so get incorporated into the larger particles that coagulation produces. In this way the coagulated precipitate takes most of the suspended matter out of the water and is then filtered off, generally by passing the mixture through a coarse sand filter or sometimes through a mixture of sand and granulated anthracite (high carbon and low volatiles coal). Coagulants / flocculating agents that may be used include:

- Iron (III) hydroxide. This is formed by adding a solution of an iron (III) compound such as iron(III) chloride to pre-treated water with a pH of 7 or greater. Iron (III) hydroxide is extremely insoluble and forms even at a pH as low as 7. Commercial formulations of iron salts were traditionally marketed in the UK under the name Cuprus.
- Aluminium hydroxide is also widely used as the flocculating precipitate although there have been concerns about possible health impacts and mis-handling led to a severe poisoning incident in 1988 at Camelford in south-west UK when the coagulant was introduced directly into the holding reservoir of final treated water.
- Polydadmac is an artificially produced polymer and is one of a class of synthetic polymers that are now widely used. These polymers have a high molecular weight and form very stable and readily removed flocs, but tend to be more expensive in use

compared to inorganic materials. The materials can also be biodegradable (Dorfner, 1991).

2.8.4 Nutrient Removal

Wastewater may contain high levels of the nutrients nitrogen and phosphorus. Excessive release to the environment can lead to a buildup of nutrients, called eutrophication, which can in turn encourage the overgrowth of weeds, algae, and cyanobacteria (blue-green algae). This may cause an algal bloom, a rapid growth in the population of algae. The algae numbers are unsustainable and eventually most of them die. The decomposition of the algae by bacteria uses up so much of oxygen in the water that most or all of the animals die, which creates more organic matter for the bacteria to decompose. In addition to causing deoxygenation, some algal species produce toxins that contaminate drinking water supplies. Different treatment processes are required to remove nitrogen and phosphorus.The removal of nitrogen is effected through the biological oxidation of nitrogen from ammonia (nitrification) to nitrate, followed by denitrification, the reduction of nitrate to nitrogen gas. Nitrogen gas is released to the atmosphere and thus removed from the water. Nitrification itself is a two-step aerobic process, each step facilitated by a different type of bacteria. The oxidation of ammonia (NH_3) to nitrite (NO_2^-) is most often facilitated by Nitrosomonas spp. (nitroso referring to the formation of a nitroso functional group). Nitrite oxidation to nitrate (NO_3^-), though traditionally believed to be facilitated by Nitrobacter spp. (nitro referring the formation of a nitro functional group), is now known to be facilitated in the environment almost exclusively by Nitrospira spp. Denitrification requires anoxic conditions to encourage the appropriate biological communities to form. It is facilitated by a wide diversity of bacteria. Sand filters, lagooning and reed beds can all be used to reduce nitrogen, but the activated sludge process (if designed well) can do the job the most easily. Since denitrification is the reduction of nitrate to dinitrogen gas, an electron donor is needed. This can be, depending on the wastewater, organic matter (from faeces), sulfide, or an added donor like methanol. Sometimes the conversion of toxic ammonia to nitrate alone is referred to as tertiary treatment. Many sewage treatment plants use axial flow pumps to transfer the nitrified mixed liquor from the aeration zone to the anoxic zone for denitrification. These pumps are often referred to as Internal Mixed Liquor Recycle pumps (IMLR pumps).

2.8.5 Phosphorus Removal

Phosphorus removal is important as it is a limiting nutrient for algae growth in many fresh water systems (for negative effects of algae see Nutrient removal). It is also particularly important for water reuse systems where high phosphorus concentrations may lead to fouling of downstream equipment such as reverse osmosis. Phosphorus can be removed biologically in a process called enhanced biological phosphorus removal. In this process, specific bacteria, called polyphosphate accumulating organisms (PAOs), are selectively enriched and accumulate large quantities of phosphorus within their cells (up to 20% of their mass). When the biomass enriched in these bacteria is separated from the treated water, these biosolids have a high fertilizer value. Phosphorus removal can also be achieved by chemical precipitation, usually with salts of iron (e.g. ferric chloride), aluminum (e.g. alum), or lime. This may lead to excessive sludge productions as hydroxides precipitates and the added chemicals can be expensive. Chemical phosphorus removal requires significantly smaller equipment footprint than biological removal, is easier to operate and is often more reliable than biological phosphorus removal. Another method for phosphorus removal is to use granular laterite. Once removed, phosphorus, in the form of a phosphate rich sludge, may be stored in a land fill or resold for use in fertilizer (Muraviev and Gorshkov, 2000).

CHAPTER THREE

3.0 MATERIALS AND METHOD

3.1 Description of the Study Site

The project was conducted on the premises of Federal University of Technology Minna, Bosso Campus, Niger State. Bosso campus (latitude 8^0 10^0N and $11^0 30^0$N, longitude 3^0 30^0E and $7^0 30^0$E) is in the southern Guinea savanna of Nigeria. The vegetation of Minna consist of scattered trees such as mango (*mangiferas indica*), shrubs : *vililaria paradox, vitex donian* and short grasses such as *Andropogon gauyamus, Brachairia bizantha, stylosanthes guyanensis, Mucuna pyruieris, Axonopus compressus Canajus cajas and Cynedon dactylon.* The climate alternate with both dry and wet season. The rainy season begins in April and ends in October. The mean monthly rainfall record from 2000 to 2010 ranges from 0.55mm to 267.7mm with March having the minimum and September having the maximum occurrence (NIMET, 2010). The dry season is marked by the influence of harmattan (tropical continental air mass) which blows across from the Sahara. It usually last from December to February. The mean monthly temperature ranges from 26.01^{0}C to 34.68^{0}C with August having the minimum record and March having the maximum record (Iyanda, 2008).

The experimental site consists of mini-channels from different parts of the school campus, these mini-channels are connected to the main school channel for discharge of domestic waste water. The main channel is about 4 km long approximately. It runs from Bosso dam through Federal university of Technology Minna, Bosso campus to Ayan- gwari area of Minna, Niger state of Nigeria. The main channel serves as source of waste water for this project.

3.2 Experimental Design of the Treatment Plant

3.2.1 Design of Treatment Plant

The treatment plant consists of three main components namely sedimentation tank, aeration tank and filtration tank. The figure below represents the flow diagram for the operation processes of domestic waste water plant. It shows basically the processes of sedimentation, aeration and filtration.

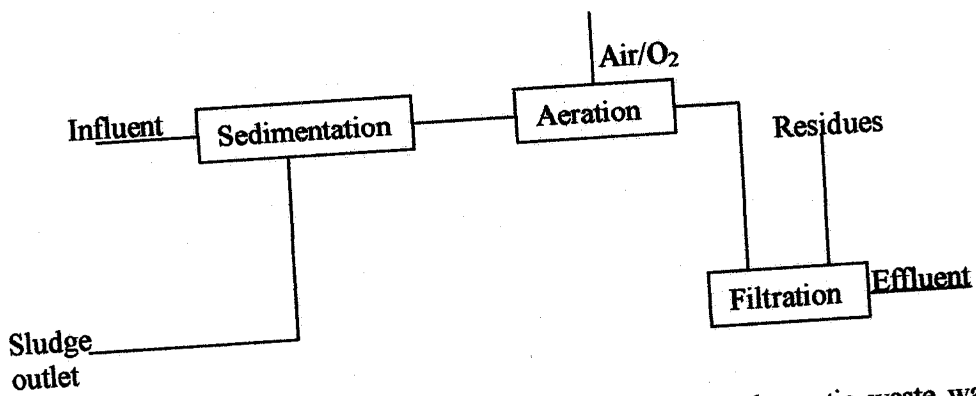


Fig 3.1. Flow diagrams for the operation processes of the domestic waste water treatment plant.

3.2.2 Experimental Procedure

Design criteria were mainly selected on the results of preliminary studies like determining settling velocities, and permeability of different grades of sands. In the course of constructing the domestic waste water treatment plant the following materials were utilized: Plastic Bucket, Union Link, Ball Valves, Elbow, Back Nut, Adaptor, Pressure Pipe, PVC Gum, Oxygen Pump, and Wooden Stand. The sedimentation, aeration and filtration tank were made of plastic. An orifice was drilled at the top-most part of the sedimentation tank for domestic waste water inlet into the tank. Another orifice was drilled at the centre for sludge outlet and

V_p = Velocity of particle (m/s)

If the density of the particle differs from that of the water, a net force is exerted and the particle is accelerated in the direction of the force:

$$F_{net} = (\rho_p - \rho)gV_P \tag{3.3}$$

Where,

ρ_p = Density of particle (Kg/m^3)

ρ = Density of water (m^3)

g = Gravity (N)

V_p = velocity of particle (m/s)

This net force becomes the driving force.

Once the motion has been initiated, a third force is created due to viscous friction. This force is called the drag force, is quantified by (Gilbert, 1998):

$$F_d = \frac{C_D A_P \rho V^2}{2} \tag{3.4}$$

Where,

F_d = Drag force (N)

C_D = Drag coefficient

A_P = Projected area of the particle (m^2)

V = Velocity (m/s)

ρ = Density of water (m^3)

Because the drag force acts in the opposite direction to the driving force and increases as the square of the velocity, acceleration occurs at a decreasing rate until a steady velocity is reached at a point where the drag force equals the driving force (Gilbert, 1998):

$$(\rho_p - \rho)gV_p = \frac{C_D A_P \rho V^2}{2} \tag{3.5}$$

Where,

C_D = Drag coefficient

A_P = Projected area of the particle (m^2)

V = Velocity (m/s)

ρ = Density of water (m^3)

g = Gravity (N)

For spherical particles (Gilbert, 1998),

$$V_p = \frac{\pi d^3}{6} \text{ and } A_p = \frac{\pi d^2}{4} \tag{3.6}$$

Thus,

$$V^2 = \frac{4g(\rho_p - \rho)d}{3C_{D\rho}} \tag{3.7}$$

Expressions for C_D change with characteristics of different flow regimes. For Laminar, Transition, and Turbulent flow, are (Gilbert, 1998):

$$C_D = \frac{24}{R_e} \text{ (Lamina)} \tag{3.8}$$

$$C_D = \frac{24}{R_e} + \frac{3}{R_e^{1/2}} \text{ (Transition)} \tag{3.9}$$

$$C_D = 0.4 \text{ (Turbulent)} \tag{3.10}$$

Where,

$$R_e - \frac{\rho v d}{\mu}$$

C_D= Drag coefficient

A_p= Projected area of the particle (m^2)

g = Gravity (N)

ρ_p = Density of particle (Kg/m^3)

V_p= Velocity of particle. (m/s)

R_e = Reynolds number

Reynolds number less than 1.0 indicate laminar flow, while values greater than 10 indicate turbulent flow. Intermediate values indicate transitional flow.

3.2.4 Design of Aeration Tank

The aeration tank is a kind of aerated lagoon. Air is supplied to the system using oxygen pump. The equipment used to deliver oxygen to the aeration system is typically provided by surface mechanical type of aerators or diffused aeration systems.

Table 3.1. Typical Aeration Tank Mixing Requirement

Type of Aeration System	Mixing Requirement
Course bubble diffuse aeration	20 to 30 scfm/1000 cu.ft.
Fine bubble diffuse	7 to 10 scfm/1000 cu.ft.
Mechanical surface aeration	0.6 to 1.15 hp/1000 cu.ft.

Source: (Hunt, 1996)

3.2.4.1 Calculation Of Oxygen Required

The amount of oxygen required for BOD oxidation in aeration basins:

$$\text{Oreq. BOD} = (\text{BODin} - \text{BODef})\ 8.34\ \text{Con. Q} \qquad (3.11)$$

$$\text{Oreq. BOD} = 1{,}668$$

Where,

BODin (mg/L) = Biochemical Oxygen Demand influent for a typical medium strength municipal waste water flow the value is (220) (Hunt, 1996).

BODef (mg/L) = Biochemical Oxygen Demand of the effluent waste water, British standard value range between 20 – 30 mg/l. For the purpose of this research 20 mg/l was used.

Con. = 1.0 Pounds of oxygen required per pound of BOD removed.

Q = 1 Municipal waste water flow in tens of gallons per day.

3.2.4.2 Mechanical Aeration Systems

To calculate the Field Oxygen Transfer Rate (FOTR) of the Mechanical Aeration System.

This equation is for the effects of surfactants, temperature, salinity, etc.

Example for Dry Season Operation: Temp. = 25^0C therefore CsT = 8.19

$$\text{FOTR} = \text{LabOTR}\left(\frac{\beta^{*} cst - cw}{cs20}\right) * \propto * \left(-(T-20)\right) \tag{3.12}$$

Dry season: FOTR = 1.27 (0_2/hp/hr)

3.2.4.3 Diffused Aeration Systems

To calculate the standard Oxygen Transfer Rate (SOTR). This equation for the effects of surfactants, temperature, salinity, membrane fouling, etc.

Dry season: Temp. = 25°C therefore Cs,T = 8.19

$$\text{SOTR} = \left(\frac{ORT}{\frac{\beta^{*} cst - cw}{cs20}}\right) \tag{3.13}$$

Dry season: SOTR = 378 (O2/hr)

Where,

Cs20 = Oxygen saturation concentration for tap water at 20^0C

CsT = Oxygen saturation concentration corrected for altitude and temperature

Cw (mg/L) = Operating dissolved oxygen concentration. Highly variable from plant to plant, 2 - 4 mg/L, is the typical range.

β = 0.95 Cs (wastewater)/Cs (tap water). This term corrects for constituents in the wastewater which impact the solubility of oxygen (Hunt,1996).

3.2.5 Design Of Filtration Tank

From Darcy's law,

$$V = \frac{Q}{A} = K\frac{\Delta H}{L} \tag{3.14}$$

Where,

K = Permeability of filter material

V = velocity (m/s)

Q = volume (m^3)

A = area (m^2)

$\frac{\Delta H}{L}$ = Hydraulic Gradient.

To design the filtration tank, the hydraulic gradient ΔH of each layer of the filter. To calculate this, the velocity of the water flows from the inlet chamber to the filter chamber will be calculated.

$$Q = LBH \tag{3.15}$$

Where,

Q = Volume (m^3)

L = Length (m)

B = Breath (m)

H = Height (m)

$$= 0.6 \times 0.4 \times 0.54$$

$$= 0.1296 m^3$$

This is the quantity of water the chamber will take for there to be a flow.

$$Q = 0.1296 m^3$$

$$\text{Velocity} = \frac{Q}{A} = \frac{0.1296}{0.5 \times 0.4} = 0.54 m/s$$

This is the velocity at which water enters the filter bed.

Assume the permeability for fine sand = 0.3×10^{-2} cm/s

$$\Delta H_o = \frac{0.54 \times 1}{0.3 \times 10^{-2}} = 18 cm$$

To find the depth of sand and gravel, we use the assumed ΔH_o,

From Darcy's law'

$$L = \frac{K\Delta H_0}{V}$$

K for fine sand $= \frac{0.3 \times 10^{-3} \times 0.18}{0.54} = 0.1m = 10cm$

K for gravel $= 2 \times 10^{-1}$ m/s

$\Delta H_o = 18cm = 0.18m$

$V = 0.54m/s$

$L = \frac{2 \times 10^{-1} \times 0.18}{0.54} = 0.06m = 6cm$

The depth of the gravel = 6cm. The filter is designed so that the depth of the fine sand is about 10cm while that of gravel is 6cm (n.d,2010).

3.3 Tank Calculation

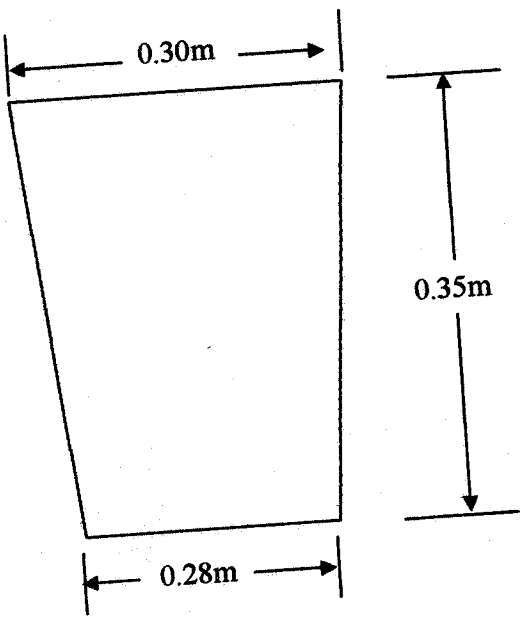


Fig. 3.4. The tank diagram showing its dimensions

$$\text{Volume (v)} = \frac{1}{3} \times \frac{22}{7} h (R^2 + Rr + r^2) \qquad (3.16)$$

Where,

Tank height (h) = 0.35m

Top radius (R) = 0.30m

Base radius (r) = 0.28m

$$V = \frac{1}{3} x \frac{22}{7} x\ 0.35\ (0.30^2 + 0.30 + 0.28 + 0.28^2)$$

$$V = 20.8 liter$$

3.4 Performance Evaluation

3.4.1 Parameters Evaluated

The major parameters evaluated by treatment plant are; Biochemical Oxygen Demanded, Chemical Oxygen Demanded, Iron, Dissolve Oxygen, Sodium, Chloride, and pH.

3.4.2 Evaluation Procedure

The sedimentation tank was filled to it capacity with the wastewater sample. It was then detained for 23 hours. After this detention period, the valve leading to the aeration tank was opened. The clean sample flow into the aeration tank. When it was filled to it capacity, the oxygen pump was activated. This pumped oxygen into the tank for three hours. After this detention period, the valve leading to the filtration tank was opened. The water sample flowed into the inlet chamber. It climbed steadily in the chamber until it started flowing into the filtration chamber. It rose in the chamber until the chamber was filled up to the brim. The filtration process proceeded. The water sample started seeping through the fine sands and gravels to the under drains. This now lead to the outlet chamber. The water was collected in a bucket outside the tank. The filtrate water was taken to the lab for post treatment analysis. Finally the sludge outlet valve was open for release of sludge.

3.5 Wastewater Treatment Plant Costing

The table below describes the material costing of the wastewater treatment plant.

TABLE 3.2. Material Costing of Treatment Unit

S/NO	Item Description	Material	Quantity	Unit cost (₦)	Total cost (₦)
1	Tanks	Plastic bucket	4	400.00	1,600.00
2	Elbow	Plastic	4	200.00	800.00
3	Adaptor	Plastic	6	200.00	1,200.00
4	Oxygen pump	Electronic device	1	4,500.00	4,500.00
5	Gum	Pvc gum	1	400.00	400.00
6	Ball valves	Plastic	4	300.00	1,200.00
7	Union link	Plastic	2	200.00	400.00
8	Tank stand	Wood stand	3	500.00	1,500.00
9	Back nut	Plastic	7	200.00	1,400.00
10	Pressure pipe	Plastic	1(6m)	500.00	500.00
TOTAL					13,500

Material = ₦ 13,500.00

Labour cost = 20% of total material cost taken to the labour cost of treatment plant.

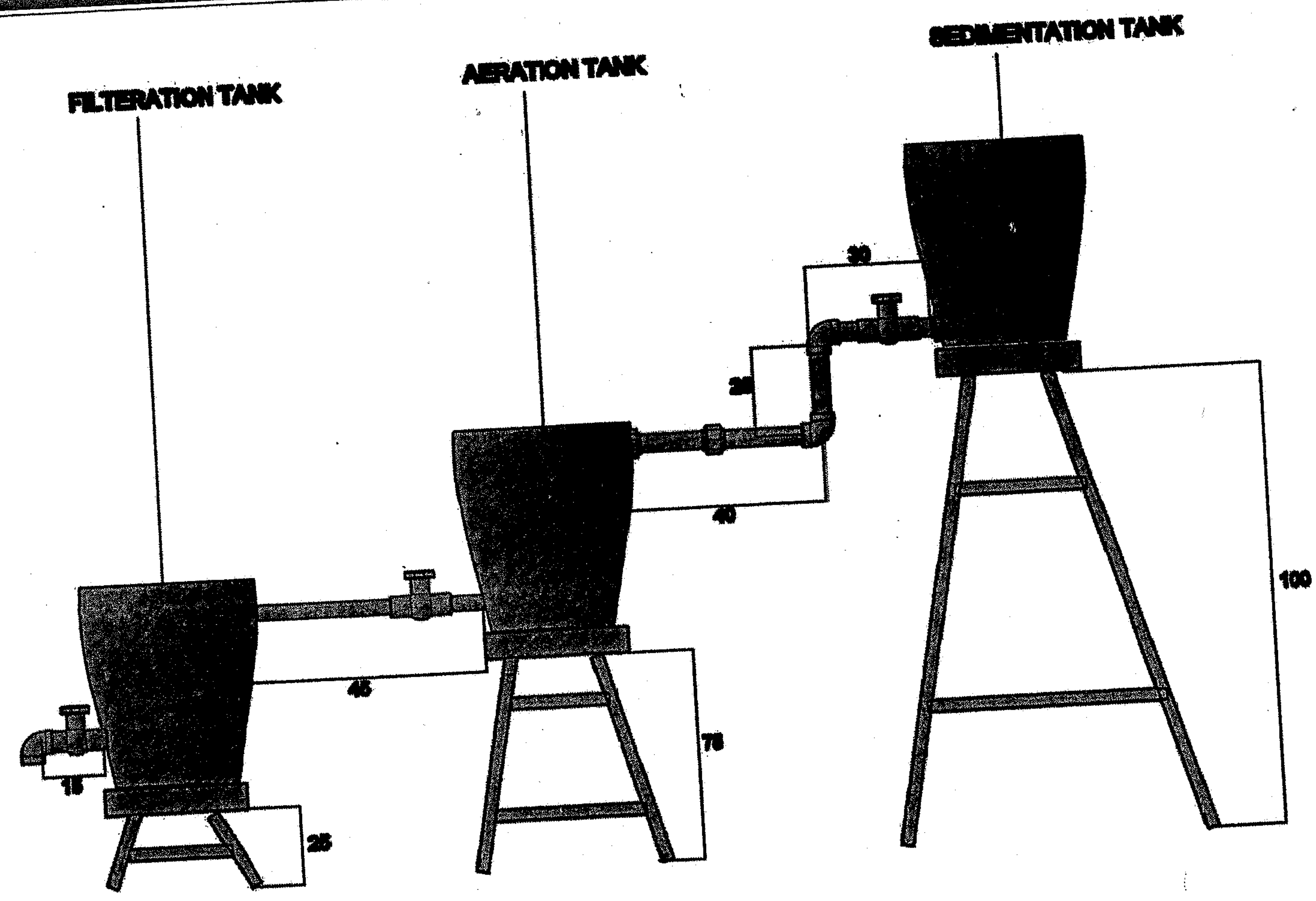
Labour cost:

$$13500 \times \frac{20}{100} - ₦\ 2,700.00$$

Cost of fabrication of the wastewater treatment plant material cost + labour cost

= ₦ 13,500.00 + ₦ 2,700

= ₦16, 200.00



ALL Dimension in (mm) Dimeter of value is 15 mm

DESIGN AND DEVELOPMENT OF DOMESTIC WATER TREATMENT PLANT FOR IRRIGATION PURPOSE	
NAME	SALAU MOHAMMED O.
MATRIC NO	2005/21667EA
APPROVED BY	ENGR. MRS H.I. MUSTAPHA

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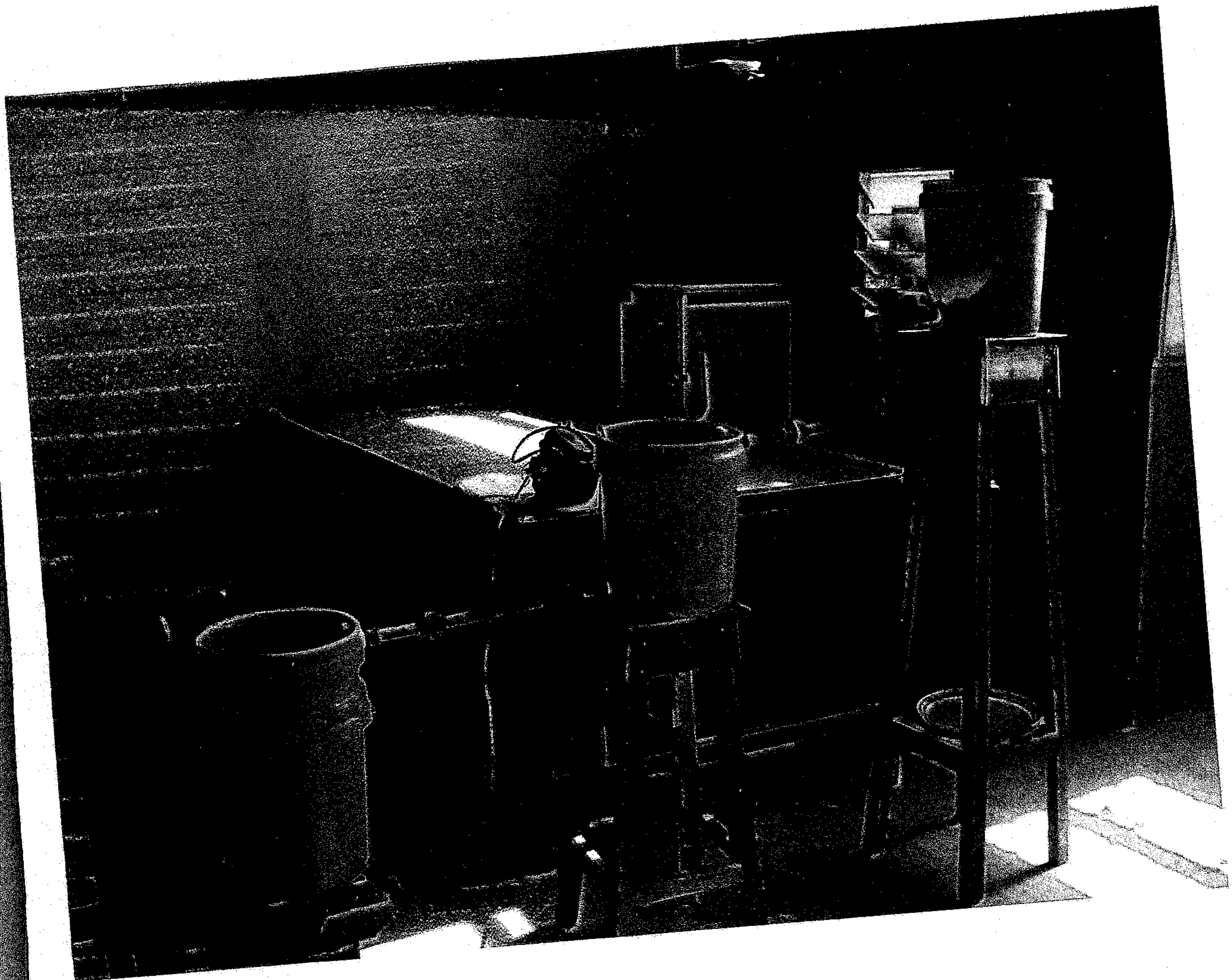

Fig 3.0 The cross section picture of the treatment plant

CHAPTER FOUR

4.0 RESULTS AND DISCUSSION

4.1 Results of Performance Evaluation

4.2 Result for Influent and Effluent Wastewater

The Table 4.1 gives the end result of the performance evaluation of the wastewater treatment plant. This includes the results of influent and effluent wastewater.

Table 4.1. Results of influent and effluent wastewater.

S/NO	Parameter	Influent Result	Effluent Result
	Physico-Chemical		
1	Bicarbonate (HCO_3)mg/l	143.6	0.00
2	Phosphate (PO_4) mg/l	5.27	3.49
3	Chloride (Cl)mg/l	165.0	0.30
4	Total Hardness mg/l	45	0.25
5	Total Dissolved solids mg/l	89.3	84.7
6	Biological Oxygen Demand (BOD) mg/l	100.0	0.4
7	Nitrate(NO_3^-)mg/l	0.65	0.22
8	Chemical Oxygen Demand (COD) mg/l	165.0	0.00
9	pH	9.16	8.4
10	Electrical conductivity (µS/cm)	598	169.3
11	Calcium (Ca)mg/l	50	0.51
12	Sodium (Na)mg/l	173	0.036
13	Potassium (K) mg/l	72	3.63
14	Magnesium (Mg) mg/l	0.30	0.24
15	Dissolved Oxygen (DO) mg/l	1.29	0.70
16	Sukphate (SO_4^{2-}) mg/l	212.5	28
17	Iron (Fe) mg/l	0.63	0.27
18	Temperature (^0C)	28	30.8
	Bacteriological		
1	Faecal Coliform (CFU/100ml)	186	14
2	E.coli (CFU/100ml)	163.0	0.00
3	Total Plate Count (CFU/100ml)	349	18.0
4	Salmonella/Shigella (CFU/100l)	0.0	0.0

4.2.1 Comparison of Treated Wastewater Results with FAO Standards for Irrigation Water Quality

The table 4.2 compares the results gotten from the treated wastewater analysis with the WHO standards for irrigation water quality.

Table 4.2. Comparing the result

S/NO	Parameter	Effluent Result	FAO Standard
	Physico-Chemical		
1	Bicarbonate (HCO_3)mg/l	0.0	1.88
2	Phosphate (PO_4) mg/l	3.49	5.0
3	Chloride (Cl)mg/l	0.30	100 – 700
4	Total Hardness mg/l	0.25	
5	Total Dissolved solids mg/l	84.7	≤1000
6	Biological Oxygen Demand (BOD) mg/l	0.40	2.0
7	Nitrate(NO_3^-)mg/l	0.22	
8	Chemical Oxygen Demand (COD) mg/l	0.00	
9	pH	8.4	6.5 – 8.5
10	Electrical conductivity (µS/cm)	168.2	< 260
11	Calcium (Ca)mg/l	0.51	1.41
12	Sodium (Na)mg/l	0.036	0.89
13	Potassium (K) mg/l	3.63	
14	Magnesium (Mg) mg/l	0.24	0.44
15	Dissolved Oxygen (DO) mg/l	0.70	2 – 7.5
16	Sukphate (SO_4^{2-}) mg/l	28.0	500
17	Iron (Fe) mg/l	0.27	5.0
18	Temperature (^{0}C)	30.3	
	Bacteriological		
1	Faecal Coliform (CFU/100ml)	18	≤100
2	E.coli (CFU/100ml)	0.0	
3	Total Plate Count (CFU/100ml)	18	
4	Salmonella/Shigella (CFU/100l)	0.0	0.0

4.3 Discussion of Results

4.3.1 Physio-Chemical Analysis

Table 4.1 shows the influent and effluent results of the parameters evaluated. The influent bicarbonate concentration was 143.6mg/l, after the performance evaluation, the concentration was reduced to (0), these may be due to the effectiveness of the sedimentation unit. The phosphate concentration in the influent wastewater was very high. It was 165mg/l, however; it was reduced to 0.30 mg/l after performance evaluation had taken place, these reductions may be due to oxygen supply in aeration unit. The influent BOD level was 100mg/l, it was reduced to 0.4mg/l. The influent COD was very high. It was 165mg/l. it was reduced to 0.0mg/l, the pH of influent wastewater was 9.16, its effluent pH was 8.24, the electrical conductivity was high. It was 598μS/cm, however, it was reduced to 169.2 μS/cm, The influent DO was 1.29 mg/l. However, instead of increasing, it was reduced to 0.70mg/l, it implies the detention time was not enough in the aeration unit (Lecture note ABE 544, 2010). The influent sulphate concentration was 212.5mg/l but it was reduced to 28.0mg/l.

4.3.2 Bacteriological Analysis

The wastewater treatment plant was effective in reducing the bacteriological characteristics of the influent waste water sample.

The influent faecal coliform was 186 CFU/100ml and this was brought down to 14CFU/100ml in the effluent waste water .The E.coli, which is directly the indicator for the pathogenic quality of the influent waste water sample, was 163CFU/100ml. it was reduced to zero in the effluent. The Total Plate Count was 349 CFU/100ml in the influent waste water sample. However, it was reduced to 18.0CFU/100ml. Table 4.2 shows the comparison between the effluent waste water parameters concentration against FAO standards for irrigation water quality.

The BOD level in the effluent was 0.04mg/l, this shows that it is acceptable because the required FAO standard says it should not exceed 2.0 mg/l, the pH of effluent was 8.4, this falls between the FAO standards of 6.5 - 8.5, the dissolved oxygen in the effluent was 0.07mg/l, this did not meet up with the FAO standards of 2.0 – 7.5mg/l, However, this is not supposed to decrease, it implies that the detention time for aeration not enough. The effluent EC was 168.2 μS/cm, this is less than the maximum recommended level of 26 μS/cm. the effluent sulphate was 28.0mg/l. this falls under the maximum recommended standard of 50mg/l. The effluent faecal coliform was 14CFU/100ml. this falls below the required maximum recommended level at 100 CFU/100ml.

4.3.3 Statistical Analysis

The statistical analysis of the treatment plant was carried out, using the Microsoft excel. These enable to know the effectiveness of the treatment plant and how efficient is the treatment plant. The result gotten from the effluent and influent of the waste water treatment plant from laboratory was used to analyse statistically and graphically. The statistical analysis also enable to plot a bar chart indicating the effluent, influent and comparison of the effluent result with FAO standard. The statistical analysis for ANOVA, Regression and Correlation are shown in Appendix A.

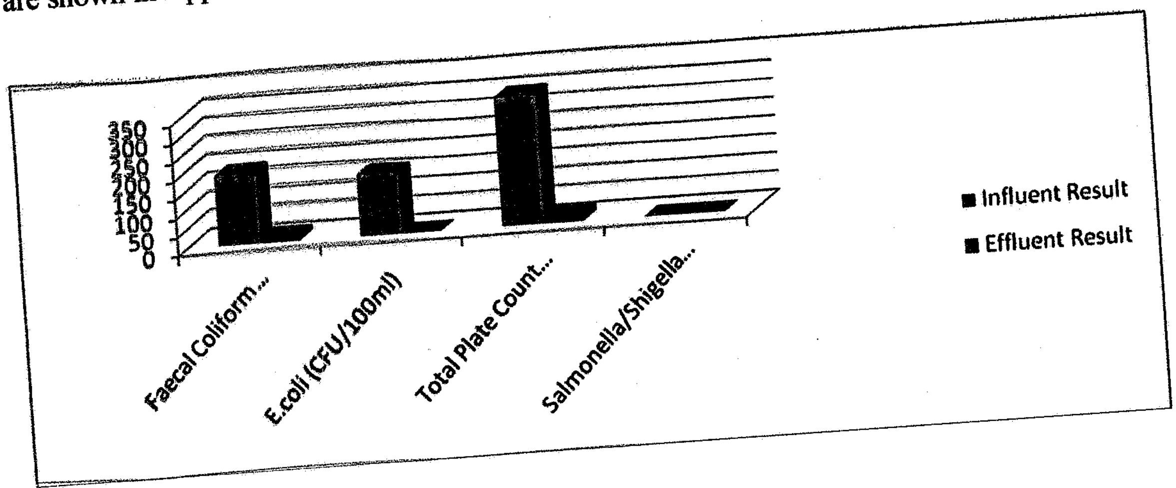

Fig 4.1 Bar chart showing the bacteriological result of the influent and effluent result

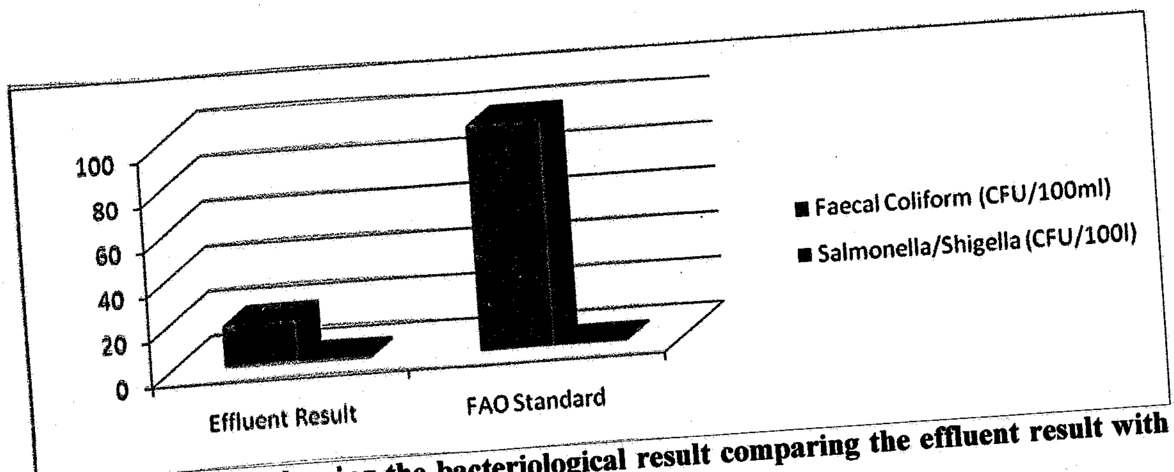

Fig 4.2 Bar chart showing the bacteriological result comparing the effluent result with FAO standard

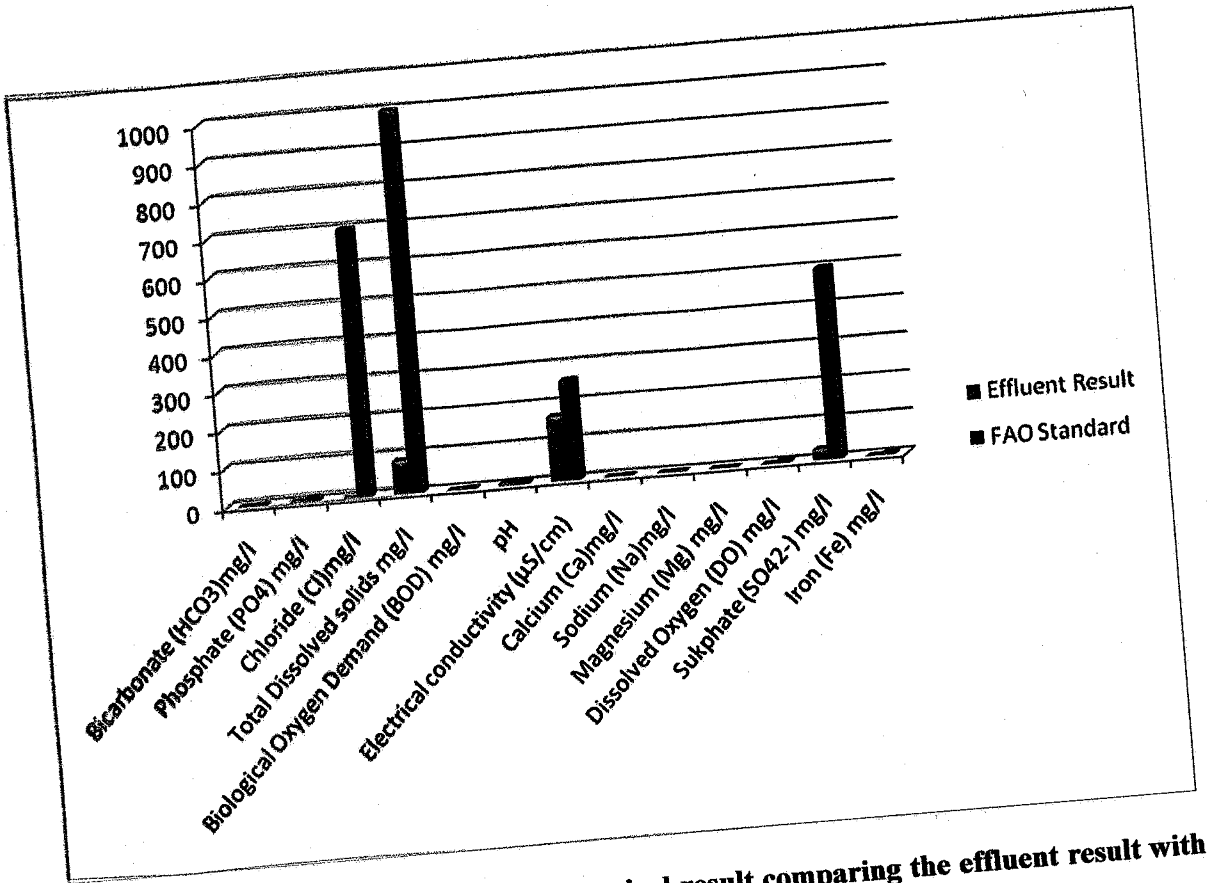

Fig 4.3 Bar chart showing the physio chemical result comparing the effluent result with FAO standard

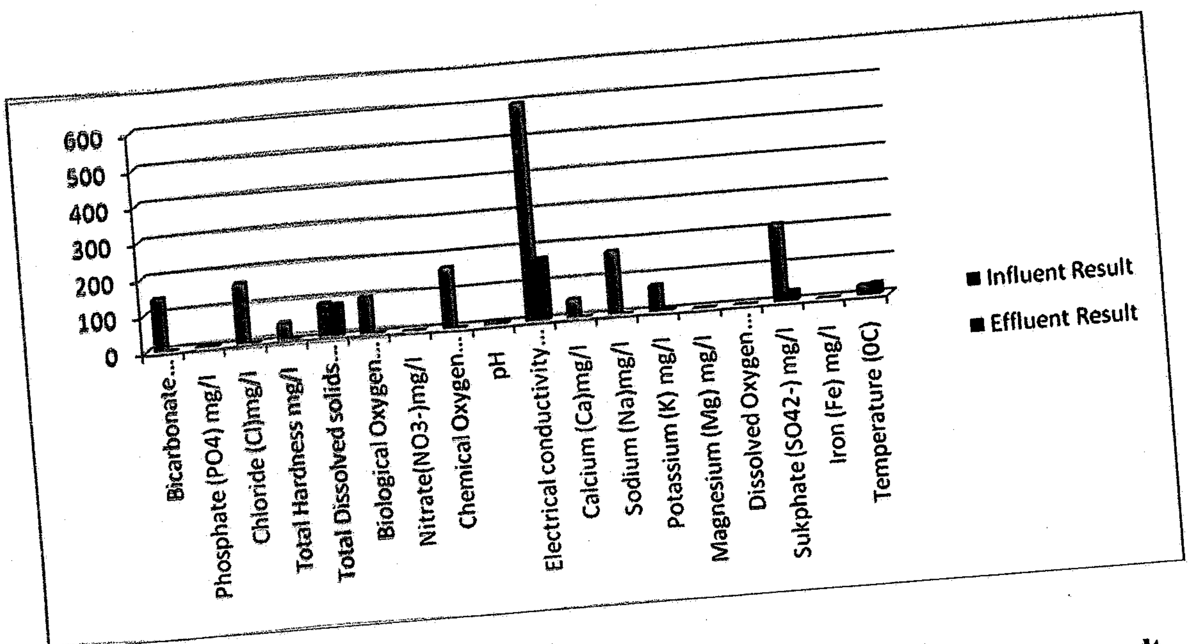

Fig 4.4 Bar chart showing the physio chemical result of the influent and effluent result

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATION

5.1 Conclusions

The scarcity of water will be one of the biggest problems in the future. To decrease this problem, new ways of water use must be executed. One possibility is the water re-use for irrigation of agricultural areas, land scape and golf courses. By choosing the irrigation of agricultural areas, the nutrients (phosphorus and nitrogen) can be used as fertilizers. Thus calculations of nutrient and water demand are inevitable. Usually the irrigation period is limited to the vegetation period. To use the nutrients in summer time and to protect the environment in winter time the wastewater treatment plant has to be operated seasonally. The conversion from ammonia to nitrate is the most sensitive biological process in the aerobic wastewater treatment process. Normally the nitrifying bacteria need several weeks to re-start full nitrification after periods without NH_4 remove. Therefore it is necessary to develop options for wastewater treatment plants, which allow a fast, re-start of the nitrification process.

The domestic waste water treatment plant was very effective in controlling the pathogenic and chemical load of the waste water. The bicarbonate concentration in influent waste water was 143.6 mg/l, it was brought down to (0), far below the required FAO standard of 1.88, The fecal coliform of the influent was 186 CFU/ml, it was brought down to 14CFU/ml, this falls under the maximum permissible concentration of 100 CUF/ml, of FAO standard. The construction was successful and completed before the evaluation, the efficiency of the waste water treatment plant was determined by using the statistical analysis method which show the effectiveness and efficiency of the treatment plant.

5.2 Recommendations

Future research projects have to develop more treatment techniques by which a seasonal operation mode can be realized. This may be a combined anaerobic/aerobic treatment process, which seems to be successful to operate wastewater treatment plant seasonally in warm climate.

The fine sand component of the filter chamber of the filtration tank should be very clean to ensure very accurate results from the use of the waste water treatment plant.

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APPENDIX A

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.821719542
R Square	0.675223006
Adjusted R Square	0.512834509
Standard Error	99.66189053
Observations	4

ANOVA

	df	*SS*	*MS*	*F*	*Significance F*
Regression	1	41300.01515	41300.02	4.158072	0.17828046
Residual	2	19864.98485	9932.492		
Total	3	61165			

	Coefficients	*Standard Error*	*t Stat*	*P-value*	*Lower 95%*	*Upper 95%*	*Lower 95.0%*	*Upper 95.0%*
Intercept	74.43939394	69.93568515	1.064398	0.398649	-226.469573	375.3483605	-226.46957	375.3483605
Effluent Result	12.50757576	6.13376535	2.039135	0.17828	-13.8838865	38.89903798	-13.883886	38.89903798

Anova: Single Factor

SUMMARY

Groups	Count	Sum	Average	Variance
Influent Result	18	1858.7	103.2611	20259.44
Effluent Result	18	331.246	18.40256	1857.095

ANOVA

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	64808.77	1	64808.77	5.860662	0.020972	4.130018
Within Groups	375981.1	34	11058.27			
Total	440789.9	35				

SUMMARY OUTPUT

Regression Statistics	
Multiple R	0.785454247
R Square	0.616938374
Adjusted R Square	0.592997022
Standard Error	90.80558064
Observations	18

ANOVA

	df	SS	MS	F	Significance F
ssion	1	212480.0692	212480.1	25.76874	0.000112241
ual	16	131930.4556	8245.653		
	17	344410.5248			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%	Lower 95.0%	Upper 95.0%
rcept	55.5196428	23.3782373	2.374843	0.030402	5.959993983	105.0792916	5.959993983	105.0792916
uent Result	2.5942847	0.51105904	5.076292	0.000112	1.51088794	3.677681459	1.51088794	3.677681459

	Influent Result	Effluent Result
Influent Result	1	
Effluent Result	0.785454247	1

APPENDIX B

TABLE B: Total Monthly Rainfall (mm), from 2000 to 2010

Year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec
2000	0.0	7.9	0.0	35.7	102.8	164.2	243.9	245.7	237.1	212.2	0.00	0.0
2001	0.3	0.0	0.0	3.6	135.9	161.0	208.8	308.5	303.0	153.4	0.00	0.0
2002	0.0	0.0	0.0	93.9	139.0	331.7	244.6	230.2	298.8	25.7	0.00	0.0
2003	0.0	0.0	5.7	98.8	42.6	201.0	143.2	226.5	260.6	180.3	0.3	0.0
2004	0.0	5.7	0.0	17.4	114.6	213	123.0	191.6	188.2	192.4	2.3	0.0
2005	0.0	0.0	0.0	32.2	151.9	210.0	211.4	241.5	77.6	0.0	0.0	0.0
2006	0.0	0.0	0.0	49.1	87.0	207.0	294.2	127.8	216.6	94.8	0.0	0.0
2007	11.2	0.0	TR	29.9	195.0	107.7	229.7	317.1	360.5	172	0.0	0.0
2008	0.0	0.0	0.4	73.1	156.6	123.9	314.0	310.1	330.2	115.1	0.0	0.0
2009	0.0	0.0	0.0	89.9	101.4	108.9	246.8	491.6	273.5	85.2	0.0	0.0
2010	0.0	0.0	0.0	46.3	131.5	109.8	260.9	248.3	230.8	161.1	13.0	0.0
Total	11.5	13.6	6.1	569.9	1358.3	1938.5	2520.5	2944.9	2776.9	1392.2	15.6	0.0
Mean	1.04	1.23	0.55	51.8	123.48	176.22	229.13	267.71	252.4	126.56	1.41	0.0

Source: Nigeria Meteorological Station Minna (NIMET,2010)