PERFORMANCE EVALUATION OF WUPA SEWAGE TREATMENT PLANT IN FCT; ABUJA

BY

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MSc/Eng/22205/12-13

A DISSERTATION SUBMITTED TO THE SCHOOL OF POSTGRADUATE STUDIES, AHMADU BELLO UNIVERSITY, ZARIA IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF A MASTER OF SCIENCE DEGREE IN WATER RESOURCES AND ENVIRONMENTAL ENGINEERING

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DECLARATION

I declare that the work in this dissertation entitled: 'Performance Evaluation of WUPA Sewage Treatment Plant in FCT'; Abuja has been carried out by me in the department of Water Resources and Environmental Engineering, Faculty of Engineering Ahmadu Bello University Zaria under the supervision of Dr. D.B. Adie and Dr. S.B Igboro. The information derived from the literature has been duly acknowledged in the text and a list of references provided. No part of this thesis was previously presented for another degree or diploma at this or any other institution.

Hassan Angyukuwi *AUDU*

________________ ________________

Signature: **Date:**

CERTIFICATION

This dissertation titled: “PERFORMANCE EVALUATION OF WUPA SEWAGE TREATMENT PLANT IN FCT; ABUJA” by Hassan Angyukuwi AUDU meets the regulation governing the award of the degree of Master of Science in Water Resources and Environmental Engineering of Ahmadu Bello University and is approved for its contribution to knowledge and literary presentation.

Dr. D.B. Adie Chairman, Supervisory Committee	________________ Signature	________________ Date
Dr. S.B. Igboro Member, Supervisory Committee	________________ Signature	________________ Date
Prof. A. Ismail Head of Department	________________ Signature	________________ Date
Prof. Kabir Bala Dean, School of Postgraduate Studies	________________ Signature	________________ Date

ACKNOWLEDGEMENTS

I would like to acknowledge and thank the following people, without whom this work could not have been completed….

Dr. D.B Adie, my Major supervisor and mentor. I appreciate God's extra grace for humility in your life, your patience, timely responses to my work and text messages, your guidance, advice, enthusiasm and constant support.

Dr. S.B Igboro, my co-supervisor who had been morally supportive during the course of this work and Prof. C.A Okuofu, for your fatherly role in bringing this work to reality. The staffers of the department of Water Resources and Environmental Engineering were all instrumental to the success of this work among who are: The HOD; Prof. Ismail A., Dr. Ibrahim F., Dr. Ajibike M.A, Dr. Adeogun B.K, Mr. Alika A.C., Mr. B.Z Tanko and Mallam Yahaya among others. Thank you all for the invaluable contribution you have made to this feast in my academic attainment.

Wupa Sewage Treatment Plant Abuja is appreciated for generously providing the wastewater, laboratory and the valuable technical knowledge use in this research. I would like to acknowledge Engr. Abdulrahaman who had graciously offered permission to use the lab and Mr. Michael Aeyini who guided me through my work at the lab.

I wish to thank and acknowledge the love of my family – Matakhitswen (my wife) and my babies (Amatsaka and Wamajiken) for their moral understanding and unparalleled support during the period of my studies. Thanks guys; I love you!!

Worthy of mentioning are Engr. Thomas Zaka Gaza and Engr. Nathaniel Zaka Gaza, Mr. Emmanuel Aji Ahmadu, Engr. Danlami Yavini Tsunatu and Mr. Danji Matsunde who have in various ways provided material assistance to this work. Without them, this project could not have been accomplished.

It was a great pleasure working with result oriented friends in the department who gave assistance to the success of this work, among whom are; Stephen James, Hassan Musa and Ahimie O.

DEDICATION

This thesis is dedicated to:

GOD, the Father,

GOD, the son,

GOD, the Holy Spirit

My beloved father

Late Mr. Audu Angyukuwi

And to my family.

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LIST OF ABBREVIATIONS

Abbreviation	Terms
APHA	American Public Health Association
BOD_5	Biochemical Oxygen Demand at 20ºC for five days
COD	Chemical Oxygen Demand
CL^-	Chloride
cfu/100ml	Colony forming units per 100 milliliters
DO	Dissolved Oxygen
et al.	et alli and others
EC	Electrical Conductivity
FC	Faecal Coliform
FCT	Federal Capital Territory
FGN	Federal Government of Nigeria
F/M	Ratio of food to microorganism
g/m^3d	Gram per cubic meter per day
kg/d	Kilogram per day
kg BOD_5/m^3d	Kilogram biochemical oxygen demand per cubic meter per day
kg BOD_5/ha d	Kilogram biochemical oxygen demand per hectare per day
HRT	Hydraulic retention time
mg/d	Milligram per liter
m^3/d	Cubic meter per day
MCRT	Mean Cell Residence Time
MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids
MLD	Million liters per day

MPN/100ml	Most probable number per 100 milliliters
NH_4-N	Ammonium as Nitrogen
NO_3-N	Nitrate as Nitrogen
NO_2-N	Nitrite as Nitrogen
NESREA	National Environmental Standards Regulation and Enforcement Agency
PE	Population Equivalent
SO_4^-	Sulfate
ºC	Degree Celsius
STP	Sewage Treatment Plant
SRT	Solid retention time
SCP	Sample collection points
TCC	Total coliform count
TSS	Total suspended solid
UNEP	United Nations Environment Program
uPVC	unplasticized Polyvinyl Chloride
µS/cm	Micro Siemens per centimeters
W.H.O	World Health Organization
WSTP	WUPA Sewage Treatment Plant
SCP 1:	Influent to the treatment plant
SCP 7C-2:	Outlet of aeration basin
SCP 8C:	Outlet of sedimentation tank
SCP 4:	Effluent outlet after UV
SP 1:	Upstream (10m) of receiving river
SP 2:	Downstream (10m) of receiving stream

ABSTRACT

The physicochemical and bacteriological qualities of effluents from WUPA Sewage Treatment Plant, Abuja were determined within the duration of twelve weeks. Grab method of wastewater sampling was used while sampling for the analysis. The sampling and analysis were carried out between May and September, 2015. Samples were collected at the inlet to the treatment plant (SCP 1), outlet of aeration tank (SCP 7C-2), outlet of sedimentation tank (SCP 8C-3), effluent outlet of the treatment plant (SCP 4), 10m upstream and 10m downstream of the receiving water body (River WUPA).

Parameters analyzed included; biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), total suspended solids (TSS), nitrate as nitrogen (NO_3-N), nitrite as nitrogen (NO_2-N), ammonium as nitrogen (NH_4-N), dissolved oxygen (DO), chloride (CL^-), sulfate (SO_4^-), faecal coliform (FC), total coliform count (TCC), pH and electrical conductivity (EC). The results of the sample analysis showed that the average removal efficiencies of the treatment plant in terms of BOD_5 and COD were 92% and 83%% respectively, while the average removal efficiency for TSS was 89%. The dissolved oxygen (DO), nitrate (NO_3-N) and nitrite (NO_2-N) concentration of the effluent were observed to have increased by an average of 46%, 61% and 72% respectively relative to the inlet values.

The average removal efficiencies for ammonium as N (NH_4-N), CL^- and SO_4^- were 58%, 19% and 30% respectively and the bacteriological contamination of the wastewater was averagely removed by 93% and 92% for faecal coliform (FC) and total coliform count (TCC) respectively. The pH and electrical conductivity (EC) were varied by 3% and 11% respectively.

The study revealed that there was no adverse impact of the effluent water on the receiving water body (river WUPA) in terms of physiochemical parameters as the treated effluent from the facility conformed to the specified discharge limits for WHO (2007) and NESREA (2011), but in terms of the bacteriological parameters; values of the total coliform count (TCC) and faecal coliform (FC) were observed to be higher than that of the river at both upstream and downstream sections, which necessitates the development of a maintenance plan for the treatment plant with emphasis on proper maintenance of the unit processes of the treatment plant and UV treating the effluent properly before discharge into the receiving water body.

A simple mathematical model was developed for BOD_5 and TSS using mathematical analysis and the relationship obtained are as shown below:

$Y = 0.4218x^2 – 19.6070x + 126.08$; where Y = BOD5 (mg/l), x = Hydraulic retention time (Hr.) and $Y = 0.6471x^2 – 23.3510x + 146.91$; where Y = TSS (mg/l), x = Hydraulic retention time (Hr.).

CHAPTER ONE
INTRODUCTION

1.1 Background

The major sources of water for human uses include but are not limited to lakes, rivers, soil moisture and relatively shallow groundwater. Three main factors that are affecting water demand over the past century are Industrial growth, Population growth and Irrigated Agriculture. Over the years water pollution has been of serious concern. The major pollutants are pathogens, nutrients, organic matter, heavy metals, toxic chemicals, silt and salt and suspended solids (Moazzam *et al.,* 2015). Particularly, South Asia and Southeast Asia are facing severe problems of water pollution. Renowned Rivers (Yellow in China, Ganges in India, and Amu and Syr Darya in Central Asia) are top on the list of the world's most polluted rivers (Sushil, 2008). The case is similar in the developing nations of the world as most rivers in the urban areas are heavily polluted with domestic sewage, industrial effluents, chemical and solid wastes (UNEP, 2002).

Industries have become integral part of modern society; as a result production of wastes is inevitable in industrial activities (Dipu, *et al.,* 2015). A material becomes waste when it is discarded without expecting to be compensated for its inherent value. Those wastes may pose potential hazard to human and the environment when improperly treated, stored, transported or disposed off or mismanaged (Misra and Pandey, 2004). Surface water bodies in developing countries are under constant threat as a result of indiscriminate discharge of polluted effluents from industrial, domestic and agricultural/sewage activities. (Kambole, 2003). Nigeria not an exception, water pollution is the most serious environmental problem due to disposal of solid and liquid wastes on land and surface water. Abundant as it may seem, water in its clean state is one of the rarest element in the world (Omole, and Longe, 2008). Like all scarce resources which have regulations guiding their exploitation, ownership, preservation and sustenance, water is protected by a body of laws, policies and regulations in order to prevent abuse FGN (2000). It is the use to which the water is to be put that determines the quality standard that must be imposed, Anyata and Nwaiwu (2002). For instance, water meant for consumption, food and pharmaceutical industrial purposes would have higher standards than water for agricultural production.

The level at which the sewage has to be treated depends on where it will be disposed and treatment standards are higher for disposal into freshwater bodies than that into the sea. However, typically even where sewage treatment plants (STPs) exist, sewage collection networks are inadequate; some portion goes for treatment and the rest flows into drains. Sometimes wastewater stagnates in pools from which it leaches into the groundwater table and contaminates underground aquifers.

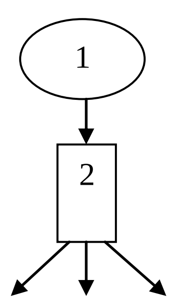
The WUPA Sewage Treatment Plant (WSTP) Abuja is an environmentally friendly project which has been designed to ensure that the ambience in the city remains clean and pleasant. It also ensures that the quality of the underground water is improved upon and the streams remain clean as only treated water is discharged back into its natural flow. The concept of the aeration basin sewage treatment plant has been lauded the world-over for its immense health and environmental benefits. It tremendously reduces the risk of epidemics associated with human waste disposal which might be traceable to the release of untreated water back to the stream and rivers.

Monitoring of the environmental parameters of the effluent would allow having at anytime, a precise idea on performance of the wastewater treatment plant and if necessary, appropriate measures may be undertaken to prevent adverse impact on the environment. The obtained result will be very much useful in identification and rectification of operational and maintenance problems. The primary cause for degradation of our water resources is the pollution caused by sewage discharged from cities and towns. Hence, the treatment plant should be routinely checked for their performance and flaws in the treatment units. This is usually carried out based on the removal efficiency of various wastewater characteristics. The WSTP adopt the biological treatment using activated sludge process. The BOD removal and degree of bacteria removal for a properly operating aerobic activated sludge treatment plant are estimated to be 80-95 and 90-95 percent respectively (Punmia *et al*., 2005).

The completion of the WUPA Sewage Treatment Plant heralded the arrival of a new dawn for the inhabitants of Abuja, capital and seat of government of the Federal Republic of Nigeria, the most populous nation in Africa. Construction works on the WUPA sewage treatment plant (WSTP) Abuja started in 2001, blazing the trail for being the largest of its kind in sub-Saharan Africa. Its state of the art, high technology and computerized accessories rank this treatment plant at par with the most modern treatment plants in the world (WSTP Abuja, 2007).

Sludge which is produced as a solid-by product of the sewage treatment process is of agricultural benefit as it could be collected and used as natural fertilizers to boost the growth and production of cash and food crops by the local farmers. The inhabitants of Abuja and the surrounding communities will continue to be thankful to the government for establishing this sewage processing plant within its domain. It epitomizes the position of government to provide a strong and virile economy by ensuring and safeguarding the environment and health of its citizens, designed to meet the requirements of 700,000 PE (Population Equivalent). The plant can accommodate an Average Dry Weather Inflow of 5,500 cubic meters per hour or 131,250 cubic meters per day. The entire treatment process can be divided into mechanical and biological phases. At no stage of the process is the addition of chemicals required. After the sewage has passed through the first stage of mechanical treatment, it enters the Aeration tanks, followed by the final clarifier where the sludge and water will be separated through a biological process. Figure 1.1 is the process layout of the WUPA treatment plant (WSTP Abuja, 2007).

The environmental impact of the WSTP and its benefits to the inhabitants of Abuja and its environs symbolizes a new approach to waste management and healthier living by the Nigerian Authorities. The completion of this project is a further testimony to the engineering capabilities of SCC Nigeria Limited and its total commitment to the overall development of Nigeria.



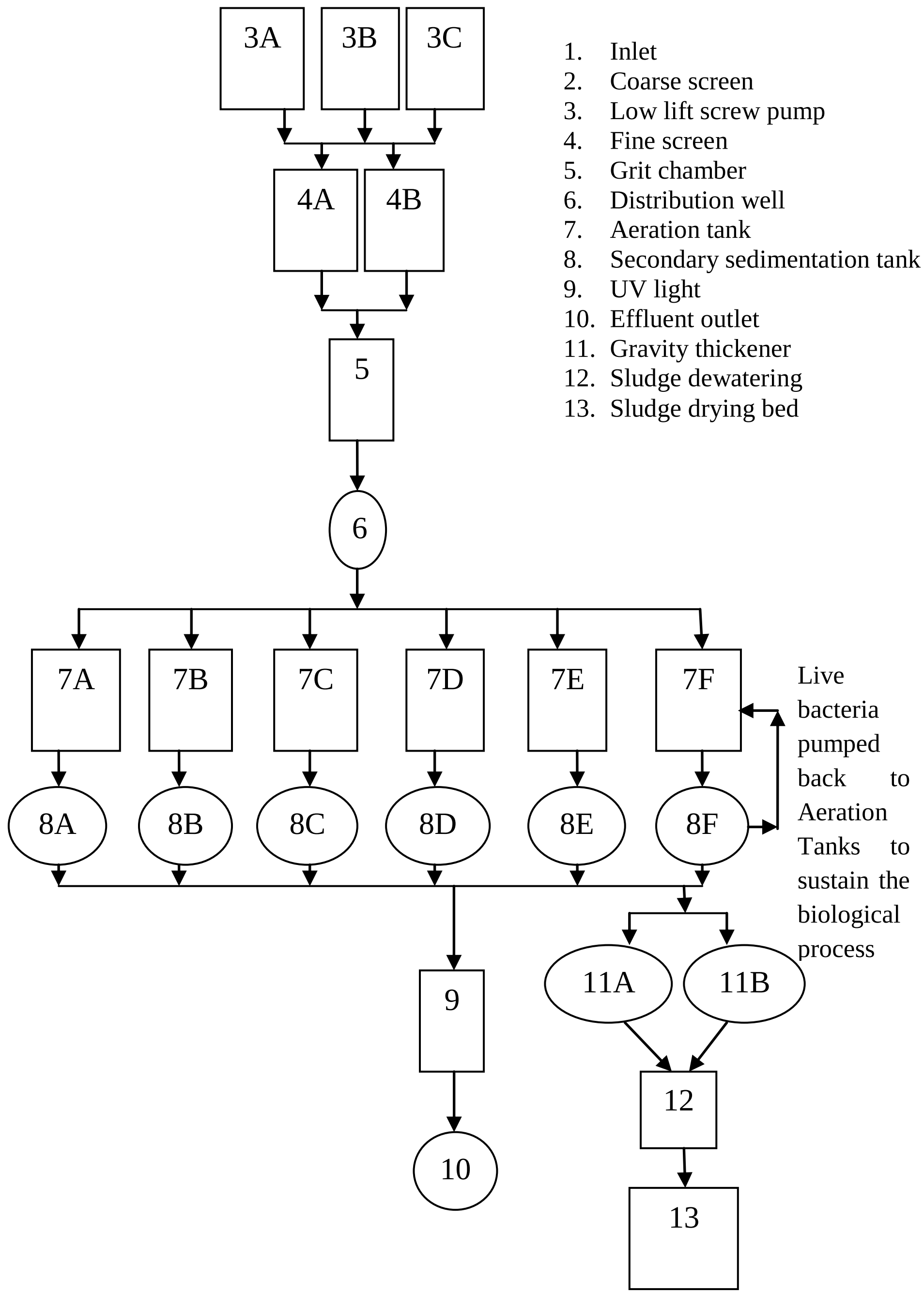


Figure 1.1: Schematics for WUPA Sewage Treatment Plant Abuja
Source: (WSTP Abuja, 2007).

1.2 Research Problem

The WUPA sewage treatment plants performance has never been determined since its commissioning in 2007, hence the need to do so. It is located upstream to the river WUPA, discharging its effluent directly into the flowing water body. The Wupa village located downstream of the treatment plant uses the river to meet its domestic water needs. Moreover there are flora and fauna as well as aquatic lives in the river, resulting in the need to ensure that the effluent quality from the treatment plant is of acceptable standards.

1.3 The study area

The Wupa Sewage Treatment Plant is located within the IDU-Industrial Area on coordinates 7˚23’N, 9˚01E next to the WUPA River. It covers an area of 297,900 square meters with surrounding rural settlement. The map showing the study area is as in Figure 1.2

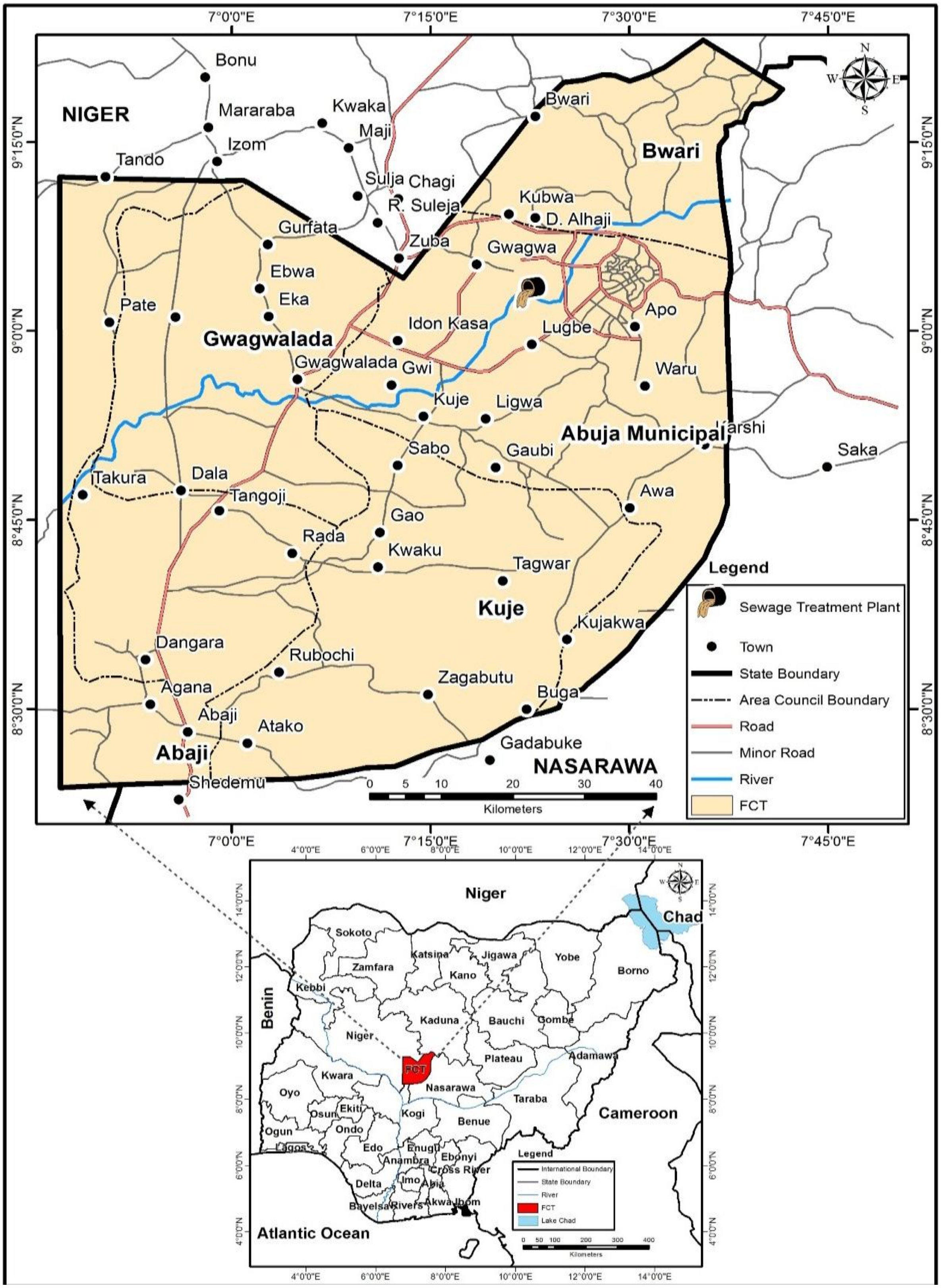


Figure 1.2: Map of FCT showing location of WSTP, Abuja

1.4 Aim and Objectives

The aim of the research is to study the Physical and Biological operations of the WUPA sewage treatment plant Abuja. This research work is undertaken to ascertain the efficiency of its performance, justifying the huge sums used in its construction.

The specific objectives are as follows:

(i) To evaluate the performance of the wastewater treatment plant in terms of biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), total suspended solids (TSS), nitrate as N (NO_3-N), nitrite as N (NO_2-N), ammonium as N (NH_4-N), dissolved oxygen (DO), chloride (CL^-), sulfate (SO_4^-), faecal coliform (FC), total coliform count (TCC), pH, and electrical conductivity (EC).

(ii) To determine the impact of the effluent from the Sewage Treatment Plant on river WUPA.

(iii) To evolve a maintenance plan for the WUPA Sewage Treatment Plant.

1.5 Project description

Effluent criteria of WSTP, BOD_5 loading, COD and average dry weather flow and volume of reactors are given in Tables 1.1, 1.2 and 1.3 respectively. The WSTP consists of six aerobic bioreactors of 27,700 m^3 capacity each, making a total of 166,200 m^3 capacity in parallel and in series to six clarifiers of 8,364 m^3 capacity each, totaling 50,184 m^3 capacity as in Figure 1.1. The steps in the treatment plant is mechanical (physical) and biological treatment processes, involving screens, grit chambers, aeration tanks with solid retention time (SRT) of 20 days, clarifiers and sludge drying bed. The Mean Cell Residence Time (MCRT) of the treatment plant is 30.4 hours.

Table 1.1 Effluent criteria for WSTP, Abuja

BOD_5	30 mg/l
Suspended solids	30 mg/l

Source: WSTP Abuja (2007)

Table 1.2 BOD_5 loading and flow to the WSTP, Abuja

Person equivalents	700,000
BOD_5	42,000 kg/d
COD	84,000 m^3/d
Flow	
Average Dry Weather Flow	131,250 m^3/d

Source: WSTP Abuja (2007)

Table 1.3 Volume of reactors and area of sludge drying bed for WSTP, Abuja.

Total land occupied by WSTP	297,900 m^2
Aeration tank – 6 units	166,200 m^3
Clarifier – 6 units	50,184 m^3
Sludge drying bed	12,100 m^2

Source: WSTP Abuja (2007)

The sewer pipelines are laid conduit and flows under gravity. They are grouped into primary, secondary and tertiary. In order to provide maximum aeration of the sewage in the system, the pipes were designed to flow half-full. It should be noted that this procedure will allow a slight safety factor for actual flows.

The minimum pipe size (used for house connections and regardless of the level of services) is 10cm, manholes have been provided at all junctions as well as at intervals of 90m. Approximately 3,000,000 meters of sewer pipes were used for the preliminary layout of the sewerage systems serving the capital city. A total of about 2, 500,000 meters of 10cm pipes were used for building connections from sewer laterals in the streets, the remaining pipe inventories required consist of 150mm to 1850mm diameter pipes.

The primary wastewater generation sources in the FCT include domestic wastewater from houses, schools and small scale industries. The sewer pipe diameters are from 100mm to 150mm uPVC. The district collector pipes (secondary sewers) are between 150mm to 300mm uPVC pipes. The remaining pipe diameters, which are the primary sewers lines (interceptor sewers) are between 350mm uPVC to 1850mm concrete pipes. Steel pipes are used to join across rivers.

1.6 Justification

Physicochemical and bacteriological parameters of effluent from a wastewater treatment plant needs to be determined in the light of the importance of the need for an acceptable quality of wastewater before being discharged into our environment. This kind of research gives opportunities to the administrators of wastewater treatment plants for identification and rectification of operational and maintenance problems (Devendra and Mahesh, 2014).

Our environment is fast degrading as a result of the discharge of toxic effluent from various industries and industrial processes to our environment and water bodies, hence the need for the construction of wastewater treatment plant and its evaluation to justify its high cost of construction and maintenance.

1.7 Scope of The Study

This research is confined to the municipal wastewater treatment at the WUPA sewage treatment plant, Abuja. The sampling and analysis is from May to September, 2015 and the literature review focused on domestic wastewater treatment using activated sludge process.

CHAPTER TWO

LITERATURE REVIEW

2.0 Introduction

The activated-sludge process was developed in England in 1914 by Ardern and Lockett and was so named because it involved the production of activated mass of micro-organisms capable of aerobically stabilizing a waste. The activated sludge is the sludge which is obtained by settling sewage in the presence of abundant oxygen so as to be supercharged with favourable aerobic micro-organisms (Prachi & Sameer, 2014). It is thus a suspended culture process with sludge return and may be either a completely mixed or a plug-flow process. The process is aerobic, with oxygen being supplied by dissolution from entrained air (Howard *et al*., 1985).

The wastewater treatment process in consideration involved two major operations which are Mechanical and Biological treatment processes.

2.1. Mechanical Treatment

Wastewater contains a wide variety of solids of various shapes, size and densities. Effective removal of these solids may require a combination of unit operations such as screening, grinding and settling. Although no material is removed by the process, flow-measurement devices are essential for the operation of wastewater treatment plants.

2.1.1 Screening

Screening devices are used to remove coarse solids from wastewater. Coarse solids consists of sticks, rags, boards and other large objects that often and inexplicably find their way into wastewater collection systems. Because the primary purpose of screens is to protect pumps and other mechanical equipment and to prevent clogging of valves and other appurtenances in the wastewater plant, screening is normally the first operation performed on the incoming wastewater. A bypass channel is provided for the smooth passage of the sewage flow, in case of clogging of the screens.

Wastewater screens are classified as fine or coarse, depending on their construction. Coarse screens usually consist of vertical bars spaced 1 or more centimeters apart and inclined away from the incoming flow. Solids retained by the bars are usually removed by manual raking in small plants, while mechanically cleaned units are used in large plants. Fine screens usually consist of woven-wire cloth or perforated plates mounted on a rotating disk or drum partially submerged in the flow, or on a traveling belt. Fine screens should be mechanically cleaned on a continual basis.

2.1.2 Grit Chamber

Municipal wastewater contains a wide assortment of inorganic solids such as pebbles, sand, silt, egg shell, glass and metal fragments. Operations to remove these inorganic will also remove some of the larger, heavier organics such as bone chips, seeds coffee and tea grounds. Together, these compose the material known as grit in wastewater treatment systems.

Most of the substances in grit are abrasive in nature and will cause accelerated wear on pumps and sludge-handling equipment with which it comes in contact. Grit deposits in areas of low hydraulic shear in pipes, sumps and clarifiers may absorb grease and solidify. Additionally, these materials are not biodegradable and occupy valuable space in sludge digesters. It is therefore desirable to separate them from the organic suspended solids.

2.1.2.1 Design criteria for grit chambers

a. **Settling velocity:** Grit chamber may be designed on a rational basis, by considering it as a sedimentation basin having discrete settling. The settling velocity for such a case is governed by the size and specific gravity of grit particles to be separated and the viscosity of the sewage. The size of separation based on the minimum size of grit to be removed is 0.2 mm although 0.15 mm is preferred for conditions where considerable amount of ash is likely to be carried by the sewage. The specific gravity may be as low as 2.4 but for design purposes, a value of 2.65 may be used. The settling velocity is given by Stoke's law applicable for particles of diameter less than 0.1 mm.

$$V_s = \frac{g(\rho_o - \rho)d^2}{18\mu} \tag{2.1}$$

Where Vs = Settling velocity (cm/s)

d = Size of particles (cm)

μ = dynamic (or absolute) viscosity of liquid (centipoise)

ρ□ = mass density of liquid (gm/cm^3)

ρ = mass density of particles (gm/cm^3)

g = acceleration due to gravity (cm/sec^2)

b. **Overflow rate:** The efficiency of grit removal can be expressed as the percentage removal of grit above a specified particle size. A grit chamber designed to remove 100% of grit particles of smallest size would also remove all grit particles larger than this. To obtain a 100% removal of the smallest size particles, it would be theoretically necessary for the detention time in the tank to equal the time required for the minimum sized particles to reach the tank bottom. It can be shown that the settling velocity Vs of the minimum sized particle is equal to the surface loading rate (Q/A), or overflow rate in order to obtain a theoretical 100% removal of the particles.

c. Detention period: The detention period for grit chambers may vary from 45 to 90 seconds. A detention period of 60 seconds is usually adopted.

d. Bottom scour and flow through velocity: The grit chamber efficiency is very much affected by the bottom scour. The scouring process itself determines the optimum velocity of flow through the grit chamber. The flow velocity should be enough to scour out the settled organic matter and reintroduce it into the flow stream.

e. Velocity control devices: The horizontal velocity of flow should be maintained constant at other flow rates also to ensure that only organic solids and not the grit are scoured from the bottom. Numerous devices have been designed in an attempt to maintain a constant horizontal velocity of flow through grit chambers in the recommended range of 15 to 30 cm/sec. A satisfactory method of controlling velocity of flow through the grit channels is by using a control section which when placed at the channel, varies the cross sectional area of flow in the section in direct proportion to the flow, such control sections may consist of the following; Proportional flow weirs, Sutro weir, Parshall flumes and Palmer Bowlus flumes.

f. **Disposal of grit:** Clean grit is characterized by the lack of odour. Washed grit may resemble particles of sand and gravel, interspersed with particles of egg shell and other similar relatively inert materials from the households. Grit washing mechanism has to be included whenever the detention time is more and flow through velocity is less. Unless washed, it may contained considerable amount of organic matter. This becomes an attraction to rodents and insects and is also unsightly and odorous. The grit may be disposed of by dumping, burying or by sanitary land fill.

2.2 Biological Treatment

The objectives of the biological treatment of wastewater are to coagulate and remove the nonsettleable colloidal solids and to stabilize the organic matter with the help of living systems which rely on mixed biological cultures. Biological treatment techniques may be classified into; attached film process, suspended growth process and combined process.

The suspended growth processes include; Activated sludge process, Aerated lagoons and Sludge digestion systems. In these processes, the micro-organisms responsible for the conversion of organic matter or other constituents in the wastewater to gases and cell tissues are maintained in suspension within the liquid in the reactor by employing either natural or mechanical mixing. The micro-organisms oxidize organic matter, in the presence of abundant quantity of oxygen in the aeration tank. Sewage is allowed to settle in a secondary sedimentation tank. This settled sludge has undergone aeration and has active microorganisms. So, some portion of this active sludge is re-circulated into the aeration tank, where the mixed liquor containing the suspended solids is under continuous agitation. The excess sludge is disposed off from the secondary sedimentation tank.

During the aeration, the microorganisms in the sewage multiply by assimilating part of the influent organic matter. In this process, part of the organic matter is synthesized into new cells and part is oxidized to derived energy. The synthesis reaction, followed by subsequent separation of the resulting biological mass and the oxidation reaction are the main mechanism of BOD removal in the activated sludge process. The biological mass generated in the aeration tank consists of zoogleal bacteria, protozoa, rotifiers e.t.c. The biomass is generally flocculant and quick settling. It is separated from the aerated sewage in a secondary settling tank and is recycled continuously to the aeration tank as an essential feature of the process.

The recycling of sludge helps in the initial build-up of a high concentration of active micro-organisms in the mixed liquor which accelerates the BOD removal. Once the required concentration of microorganisms in the mixed liquor has been reached so as to maintain proper food/micro-organism ratio (F/M) for optimum operation, its further increase is prevented by regulating the quantity of sludge recycled and wasting the excess from the system.

The BOD removal is evaluated based on the BOD_5 of the aeration tank influent and the BOD_5 of the final effluent after sludge separation. The activated sludge process is truly an aerobic process since the biological floc is suspended in a liquid media containing dissolved oxygen. Aeration conditions must be maintained in the tank. However, in the final clarifier, the dissolved oxygen concentration can become extremely low. Dissolved oxygen extracted from the mixed liquor is replenished by air supplied to the aeration tank. The micro-organisms require food, oxygen and nutrients and yield carbon dioxide, water and new micro-organisms, as indicated by equation (2.2).

$$\text{Organic Material} + O_2 + \text{Nutrients} \xrightarrow{\text{Microorganisms}} CO_2 + H_2O + \text{Energy} + \text{Microorganisms} \qquad (2.2)$$

The total oxidation of organics in the sewage by micro-organisms takes place in two phases. The first phase involves the conversion of organic matter to CO_2, water or new micro-organisms. The second phase involves endogenous respiration during which process the micro-organisms consume their own cell materials for energy. The endogenous respiration rate is relatively slow, and at the end of this phase a non-biodegradable residue remains. At this point the organic compounds are considered to be oxidized and are no longer pollutants. Carbonaceous compounds are oxidized first and on continuation of the process, the nitrogenous materials are converted into nitrites and nitrates.

2.2.1 Aeration Tank

Aeration is the most important operation in the activated sludge process, so as to provide oxygenation and mixing. The aeration facilities are designed to meet the calculated oxygen demand of the process while maintaining in the aeration tank a minimum dissolved oxygen (DO) of about 1-2 mg/l which is necessary for proper development of biological sludge.

Aeration and dissolved oxygen has two main objectives: keep the oxygen concentration within the appropriate limit, usually 2 mg/l, to maintain the microorganisms active, and ensure that the tank contents are sufficiently well mixed to keep the solids in suspension. Low dissolved oxygen concentration can limit the growth of microorganisms and encourage the predominance of filamentous bacteria with the subsequent deterioration of the effluent quality. On the other hand, high dissolved oxygen concentration represents a high energy waste through excess turbulence, especially with mechanical aerators, that may break up the biological floc resulting in poor settling characteristics and high concentration of solids in the effluent.

In addition to supplying dissolved oxygen, the aeration devices have also to provide adequate mixing and agitation so that the mixed liquor suspended solids (MLSS) do not settle down. In an activated sludge process, three methods are employed for the process of aerating the sludge viz; Diffused air aeration, Mechanical aeration, and combined diffused air and Mechanical aeration.

2.2.1.1 Aeration Period

The aeration period is the detention time of the raw-wastewater flow in the aeration tank, expressed in hours. The period of aeration depends upon the following; Strength of sewage and mass liquor suspended solid (MLSS) concentration, desired degree of purification in terms of BOD removal, rate of aeration and proportion of return activation sludge.

According to Punmia *et al*., (2005), a number of empirical formulae and charts are available to determine the aeration period. The following are two empirical formulae expressed as equation (2.3) and (2.4) respectively.

a. American Public Health Association Formula;

$$T = \frac{L_a}{20} - 1 \qquad (2.3)$$

Where T = aeration time (hours) and

L_a = BOD of the aeration tank sewage influent (mg/l) to be removed.

b. M/S Ames Crosta Mills and CO. Ltd. (England) formula;

$$T = \left(\frac{L_a}{10}\right)^{\frac{3}{4}} \qquad (2.4)$$

For complete treatment (such that the effluent is fairly stabilized with the presence of nitrates and some dissolved oxygen) a period of 4 to 6 hours is required in America, 10 to 12 hours in Britain and 6 to 10 hours in India. A noteworthy feature of the activated sludge process is the rapidity with which organic matter is oxidized when the sewage is first brought in contact with the active sludge.

2.2.1.2 Volume of return activated sludge

The volume of return activated sludge from the secondary clarifier to the aeration tank mainly depends upon the extent of the BOD desired to be removed. Table 2.1 gives the volume of activated sludge to be added to remove the desired BOD.

Table 2.1 Volume of Return Activated Sludge

Desired BOD removal (ppm or mg/l)	Percentage of Return Activated Sludge (%)
150	25
250	30
300	35
400	40
500	48
600	53

Source: Punmia *et al*., (2005)

2.2.1.3 Capacity of aeration tank

The capacity (V) of aeration tank depends upon on the following factors; Aeration period, Volume of Returned Sludge and Volume of flow of sewage.

The relationship in equation (2.5) gives the capacity of an aeration tank in m^3

$$V = (Q + Qs)\frac{T}{24} \qquad (2.5)$$

Where V = Capacity of aeration tank (m^3)

Q = Volume of flow of sewage (m^3/day)

Qs = Volume of returned activated sludge (MLD)

T = aeration period (hours).

2.2.1.4 Aeration Tank Loading Criteria

The loading rates of aeration tank are based on the following criteria;

a. Hydraulic retention time (HRT) or aeration period: The aeration period or loading rate expresses the rate which sewage is applied in the aeration tank. A loading parameter that has been developed empirically over the years is the hydraulic retention time (HRT) which is expressed as equation (2.6)

$$\text{HRT (hours)} = \frac{V}{Q \times 1000} \times 24 \tag{2.6}$$

Where V = Volume of aeration tank (m^3)

Q = Sewage inflow, (m^3/d) (excluding sludge recycle).

b. Volumetric BOD_5 loading: The volumetric BOD_5 is another empirical loading parameter which is defined as the BOD_5 applied per unit volume of aeration tank, expressed as equation (2.7)

$$\text{Volumetric load (kg } BOD_5/m^3) = \frac{Q \times L_a}{V} \tag{2.7}$$

Where L_a = Influent BOD_5 to aeration tank (mg/l).

c. Organic loading based on food to micro-organisms ratio (F/M ratio): It is an important organic loading criterion in which BOD loading (representing food F to the micro-organisms) is expressed with regard to the microbial mass M (represented by MLSS in the aeration tank). The organic loading rate is defined as the ratio of kg BOD_5 applied per day (representing microbial food) to kg MLSS in aeration tank (representing micro-organisms), expressed equation (2.8)

$$F/M = \frac{Q \times L_a}{(V/1000)X_t} \tag{2.8}$$

Where X_t = Mixed Liquor Suspended Solids (MLSS), mg/l

The F/M ratio is the main factor controlling BOD removal. Lower the F/M value; the higher will be the BOD removal in the plant. The F/M ratio can be varied by varying the MLSS concentration in the aeration tank.

d. Solids Retention Time (SRT) or Mean Cell Residence Time (MCRT) or Sludge Age:

Solids Retention Time (SRT) is another parameter that could be used for checking the design of activated sludge systems, it is also known as mean cell residence time (MCRT) or sludge age (θc) expressed as equation (2.9)

$$\theta c = \frac{X}{\Delta X/\Delta t} \tag{2.9}$$

Where; X = total microbial mass in a reactor

$\Delta X/\Delta t$ = total quantity of solids withdrawn daily including solids deliberately Wasted and those in the effluent.

The mean cell residence time (θc) or sludge age may be defined as average time for which the mass of suspended solids (or the biological solids) remain under aeration. Though the hydraulic retention time may be only few hours, the residence time of biological solids is much greater and while the sewage (liquid) passes through the aeration tank only once within the hydraulic retention time (HRT), the resultant biological growth and the extracted organic solids are repeatedly recycled from the secondary settling tank back to the aeration tank thereby increasing the retention time of solids known as solids retention time (SRT) or mean cell residence time (MCRT).

The studies by Haydar, *et al*., (2007) showed that various parameters that affect the performance of activated sludge process relating to growth of microorganisms and substrate utilization on which the operation of the reactor is based include mean cell residence time (θc) in days, mixed liquor volatile suspended solids (MLVSS) concentration expressed as X in mg/l, hydraulic detention time (θ) i.e aeration time in hours, food to microorganism (F:M) ratio in kgBOD$_5$/kg MLVSS/day and the dissolved oxygen (DO) in mg/l in the reactor.

2.2.1.4 Index of the mass of active micro-organisms:

The Mixed Liquor Suspended Solids (MLSS) content generally taken as an index of the mass of microorganisms in the aeration tank. However the MLSS will contain not only active microorganisms but also their dead cells as well as inert organic and inorganic matter derived from the influent sewage. Composition of MLSS in the aeration basin is determined by equation (2.10)

$$X_t = X + X_{ii} \tag{2.10}$$

Where

X_t = MLSS, (mg/l)

$$X = \text{MLVSS, (mg/l)} = X_a + X_e + X_i \tag{2.11}$$

X_a = active microbial mass, mg/l of VSS

X_e = endogenous respiration mass, mg/l of VSS

X_i = inert, non-biodegradable organic suspended solids, mg/l of VSS

X_{ii} = inert, inorganic suspended solids, mg/l of non-volatile SS

The Mixed Liquor Volatile Suspended Solids, (MLVSS) value is generally used and is preferable to MLSS as it eliminates the effect of organic matter.

As opined by Punmia *et al*., (2005) the suspended solids concentration (MLSS) maintained in the mixed liquor of a conventional activated sludge process ranges from 1,500 to 3,000 mg/l.

2.2.2 Secondary clarifier

The biomass generated by secondary treatment represented a substantial organic load and must be removed to meet acceptable effluent standards. A typical flow diagram of an activated sludge process treatment plant is as shown below;

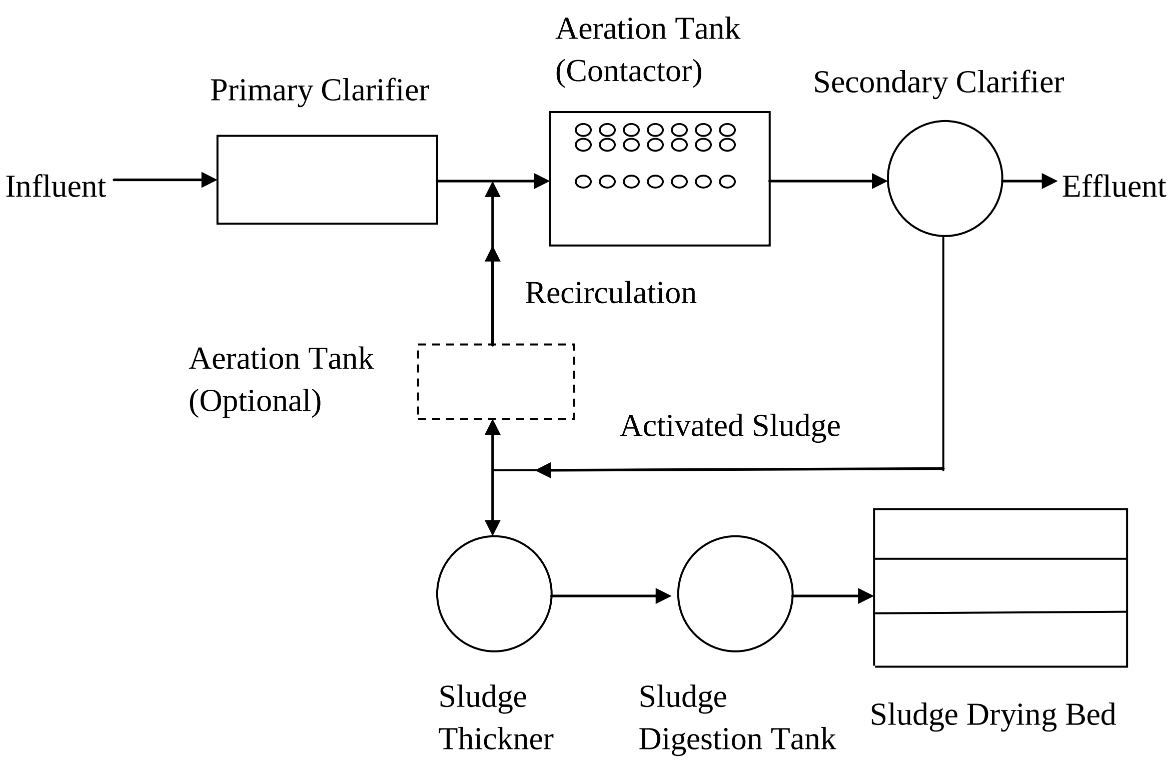


Figure 2.1 The schematic flow diagram of activated sludge process
Source: Punmia, *et al*., (2005).

The secondary clarifier (SC) is an integral part of the activated sludge system. It has two main functions: it separates the biomass from the water in order to produce a good quality effluent free from settleable solids and it also thickens the biomass. Part of the thickened biomass is then wasted as sludge and part of it is returned to the biological reactor to maintain an appropriate biomass concentration. The SC also removes floating foam and scum produced in the aeration tank (Gerardi and Wiley, 2002; Spellman 2003).

The operation of the secondary clarifier is crucial for the whole treatment plant (Gerardi and Wiley, 2002; Chen, 1993). As Beck (1984) puts it "it is in the secondary clarifier where adverse operational problems of bulking, rising, or dispersed sludge either develop or become critically apparent". The term "bulking sludge" refers to sludge that has poor settling characteristics and poor compactability. Causes of sludge bulking include the growth of filamentous organisms or bacterial cells swelling through the addition of water. "Rising sludge" is caused by the denitrification in the secondary clarifier. Denitrification may result in nitrogen gas becoming trapped in the sludge layer and causing the sludge to rise. Another operational problem present in the absence of filamentous organisms is "dispersed sludge" which thickens easily but gives an effluent with high concentration of fine suspended solids. Hence, the main goal in the operation of the secondary clarifier is to prevent excessive rise of the sludge blanket, which eventually may result in loss of sludge into effluent. This not only increases the effluent concentration of solids and organic matter considerably, it also affects the performance of the activated sludge process itself, since biomass which is necessary in the aeration tank for proper functioning of the process is lost from the system (Rabee, 2009).

2.3 Case Study on Activated Sludge Process Treating Domestic Wastewater

Chennai district, Tamilnadu, India; at present has six sewage treatment plants with an overall treatment capacity of 264 MLD and estimated generation of 158 MLD. The treatment system adopted activated sludge process for treating wastewater. The volume of the aeration tank was 20,184 m^3 and the volumetric flow rate was 6.25 m^3/min.

Namasivayam *et al*., (2014), studied the performance of the sewage treatment plant (STP) based on the removal efficiency of BOD, COD, TDS and TSS. Wastewater was collected from the inlet tank, aeration tank and final outlet and was characterized for pH, dissolved oxygen (DO), mixed liquor volatile suspended solids (MLVSS), mixed liquor suspended solids (MLSS), total dissolved solids (TDS), total suspended solids (TSS), chemical Oxygen demand (COD), biochemical oxygen demand (BOD), chloride and sulfate as per standard methods of wastewater analysis.

BOD of wastewater in the inlet tank varied from 250 mg/l to 290 mg/l, 23 mg/l to 40 mg/l in the aeration tank and final outlet BOD was 12 mg/l. BOD removal during the study varied from 90% to 95% and the treatment system was able to achieve a maximum BOD removal of 95.86% (Namasivayam *et al*., 2014). TSS removal of the domestic wastewater in the various units of STP also showed a reduction from 553 mg/l to 23 mg/l. A maximum removal efficiency of 95.68% was observed during the study. There was slight reduction in the TDS of the wastewater in the various treatment units of the STP compared to other parameters. TDS of the wastewater in the various treatment units varied from 1,600 mg/l to

2,700 mg/l in the inlet tank, 1,100 to 200 mg/l in the outlet. A maximum removal efficiency of only 55.56% was observed (Namasivayam *et al.*, 2014).

Ukpong (2013) studied the performance of an activated sludge wastewater treatment plant (ASWTP) at QIT, Ibeno Local Government Area of Akwa-Ibom State, Nigeria. The objective of the study was to evaluate the performance of the wastewater treatment plant in order to investigate the influent and effluent composition of the wastewater and to compare the level of compliance of the parameters to the WHO/DPR standards, as well as to determine the effectiveness of the sewage treatment plant.

QIT is a crude oil terminal of Mobil Producing Nigeria Unlimited (MPNU), a subsidiary company of Exxon Mobil located in Ibeno Local Government Area, Akwa-Ibom State. Qua Iboe River Estuary which lies within the study area has Douglas Creek emptyng into it, the point where petroleum exploration and production (E & P) waste from the Exxon Mobil Qua Iboe Terminal (QIT) tank farm are transferred to the lower Qua Iboe River Estuary and adjourning creeks through two 24" diameter pipes (Akpan, 2003).

The sewage treatment plant is a factory packed steel unit manufactured by PurcStream, Inc. Florence, USA. The system is based on the extended aeration method of sewage treatment, consisting basically of four operations, viz; Screening, Aeration, Settling and Chlorination.

Samples were collected into sterilized plastic containers measuring about three litres and were clearly labeled for identification and placed in ice until arrival at the laboratory where it was stored in refrigerators at 2˚C before analysis. Physical and chemical parameters including BOD_5, COD, TDS, pH, odour, colour, temperature, Oil and Grease and Total

Coliform per 100 ml was analyzed according to the standard method as described by (APHA, 1998).

The mean characteristics of the raw wastewater were BOD_5 250mg/l, COD 140mg/l, DO 10 mg/l, Oil & grease 9.0 mg/l, Total coliform per 100 ml 1,000, Total Alkalinity 152 mg/l, TDS 8,600 mg/l, TSS 4,000 mg/l, pH 5.6 and Temperature 30.5˚C. The quality of the treated effluent from the treatment units was observed to have reduced considerably. BOD_5, COD, DO, Oil & grease and Total coliform per 100 ml were 70, 65, 4, 0.05 and 170 mg/l respectively with average percentage reduction of BOD_5 72.0%, COD 53.57%, DO 60%, Oil & grease 99.44% and Total coliform 83%. The TDS 2700 mg/l recorded a percentage reduction of 68.6%, TSS, pH and Temperature were 45 mg/l, 7.01 and 28.5˚C respectively, the percentage reduction was TSS 99.9%, pH 20.1% and Temperature 6.56% (Ukpong, 2013).

Ahmed and Mahmoud (2014) studied the feasibility of using activated sludge process for the treatment of tannery wastewater. The activated sludge reactor, used for the study, was developed in the laboratory. The lab scale model comprised of primary sedimentation tank having volume of about 5.0 L, aeration tank having volume of about 5.0 L and final clarifier having volume of about 5.0 L was used for the treatment study. The aeration tank was operated continuously for 120 hours of contacts. Primary and secondary clarifiers were operated for 45 minutes. The secondary clarifier was equipped with a trough to collect the effluent.

Samples of wastewater from contaminated sites with tannery wastewater - chromium stage – Elmontazah tannery, Ain El Sira, Cairo, Egypt, were collected, analyzed within 8hr. and stored in a refrigerator at 4ºC. Activated sludge was collected from a nearby drain from Zenin wastewater treatment plant. The pH of tannery wastewater was firstly adjusted using 0.1 M HCL and 0.1 M NaOH. The reactor was fed with tannery wastewater in primary sedimentation tank for 45 min then transferred to the aeration tank for 120 hour, and transferred to the final clarifier for 45 min. The whole reactor content was kept under aerobic condition by supplying adequate air from aqua pumps, airflow capacity of 1.33 L/min. Various parameters like pH, MLSS, Total suspended solids (TSS), sludge volume index (SVI), COD, BOD, chromium, ammonia nitrogen, phosphorous, oil & grease, Turbidity, dissolved oxygen concentration (DO) and temperature of the reactor were monitored regularly under different hydraulic retention time (HRT).

It was evident from the results that the adopted HRTs exerted significant effect on the reactor performance in terms of COD, BOD, Turbidity, TDS, TSS, Ammonia, Phosphorous and Oil & Grease. At HRT of 2hr, the removal efficiency was 53.3% and 37.5% for COD and BOD, respectively. At HRT of 72hr, the removal efficiency was 83.8% and 82.7% for COD and BOD, respectively. At HRT of 120 hr, the removal efficiency was 98.4% and 98.3% for COD and BOD, respectively. Turbidity of tannery wastewater influent was higher with a value of 540 NTU indicating higher solids and organics. At HRT of 2hr, the removal efficiency was 24.1%, 4.2% and 28.8% for turbidity, TSS and TDS, respectively. At HRT of 72hr, the removal efficiency was 84.3%, 75.4% and 86.8% for turbidity, TSS and TDS, respectively. At HRT of 120 hr, the removal efficiency was 99.1%, 99.9% and 98.8% for turbidity, TSS and TDS, respectively. At HRT of 2hr, the removal efficiency was 20.96%, 24.9%, 33.6% and 38.1% for ammonia nitrogen, Phosphorous, Oil & greases and chromium, respectively. At HRT of 72hr, the removal efficiency was 86.3%, 85.4%, 86.8% and 89.9% for ammonia nitrogen, Phosphorous, Oil & greases and chromium, respectively. At HRT of 120 hr, the removal efficiency was 98.8%, 98.6%, 99.1% and 99.3% for ammonia nitrogen, Phosphorous, Oil & greases and chromium, respectively (Ahmed and Mahmoud, 2014)

CHAPTER THREE

MATERIALS AND METHODS

3.1 Materials and Reagents

1. Digester or Thermoreactor (Spectroquant TR320)

2. Spectrophotometer (Spectroquant NOVA 60)

2. Burette

4. 250 ml conical flask

5. Micro Pipettes (eppendorf Research 1000)

6. Pipette (eppendorf Research 5000)

7. Tissue papers

8. Filter paper (Whatman 110 mm ϕ Cat No 1001110)

9. Measuring bottle

10. Electrically heated temperature controlled oven (Binder)

11. Evaporating dish

12. Timer

13. Measuring cylinder (100mL and 250ml)

14. Plastic bottle for collection of sample

15. EMB Plates

16. Neogloves (Surgical gloves)

17. Pressure pump (Model No DOA-P730-NB)

18. Beakers

19. Funnel

20. Oxygen reaction bottle

21. BOD bottle (Oxitop)

22. Sampling device for collection of sample (Sampler)

23. Magnetic stirrer (Heidolph instrument MR 1000)

24. Laminar flow cabinet

25. Autoclave (Certoclav sterilizer GmbH, A-4050 Traun/Australia)

26. Incubator

27. Desicator

28. Digital weighing balance (Model EP114C SNR1126372109 OHAUS)

29. Distilling flask, 125mL

30. Water bath

31. pH meter (pH 330i)

32. Test tubes

33. Petri dish

34. Reaction cell

35. Rapping fold

36. Litmus paper

37. Mercuric sulphate

38. Starch indicator (Solution)

39. Magnesium (O_2 -1)

40. Oxygen (O_2 - 2)

41. Ammonia reagent (NH_4^- -1, NH_4^- -2 & NH_4^- 3)

42. Nitrate reagent (NO_3^- -1)

43. Nitrate pellet (NO_3^- -2)

44. Chloride reagent (Cl^- -1, & Cl^- -2)

45. Potassium dichromate

46. Sulfuric acid (H_2SO_4)

47. Ferrous ammonium sulphate

48. Silver Sulphate

49. Standard sodium thiosulphate

50. Mackonkey agar

51. 15 tubes of double strength lactose broth

52. Barium chloride crystal

53. Sodium hydroxide

54. Distilled water

55. Sodium hydroxide pellet

56. EDTA reagent

57. Standard ammonium solution

58. Hydrochloric acid solution

59. Aluminium hydroxide suspension

60. Standards silver nitrade solution

61. Potassium chromate indicator solution

62. Magnifying lens

63. Test tube rack

64. EC Broth

65. Maccarney bottles

66. Measuring tape

3.2 Research Method and Data Collection

Quantitative analysis was used for the performance evaluation of the wastewater treatment plant being carried out at the Treatment Plant's laboratory Abuja. Grab method of wastewater sampling was used.

3.2.1 Wastewater sample

Primary and Secondary data were used in the analysis. For the sake of evaluating the efficiency of the WWTP, samples were collected by grab on weekly basis from four points (Fig. 3.1) along the treatment plant and two points at upstream and downstream of the receiving river. Each sampling point was sampled 12 times. Since it was not possible to take composite samples, therefore sufficient grab samples were collected to arrive at an average performance value for the plant.

For secondary data, result of physicochemical and bacteriological analysis was collected from the office of the quality control officer for period of January, 2014 to December, 2014 covering for both the rainy and dry seasons.

3.2.2 Sampling plan

Samples of wastewater were collected from the treatment plant for analysis as indicated by sample collection points (SCP). Samples were collected at the influent to the treatment plant (SCP 1), outlet of aeration tank (SCP 7C-2), outlet of the sedimentation tank (SCP 8C-3), the effluent outlet after the UV (SCP 4), 10m upstream (SP 1) and 10m downstream (SP 2) of the receiving water body (River WUPA).

During the period of the study at the WSTP, it was found that the plant was operating at 30 percent capacity hence, the low lift screw pump 3A and 3B, Aeration Tanks 7A and 7B and the Secondary Sedimentation Tanks 8A and 8B were not in operation because of the low volume of wastewater to the treatment plant, though they are used interchangeably as the treatment progresses.

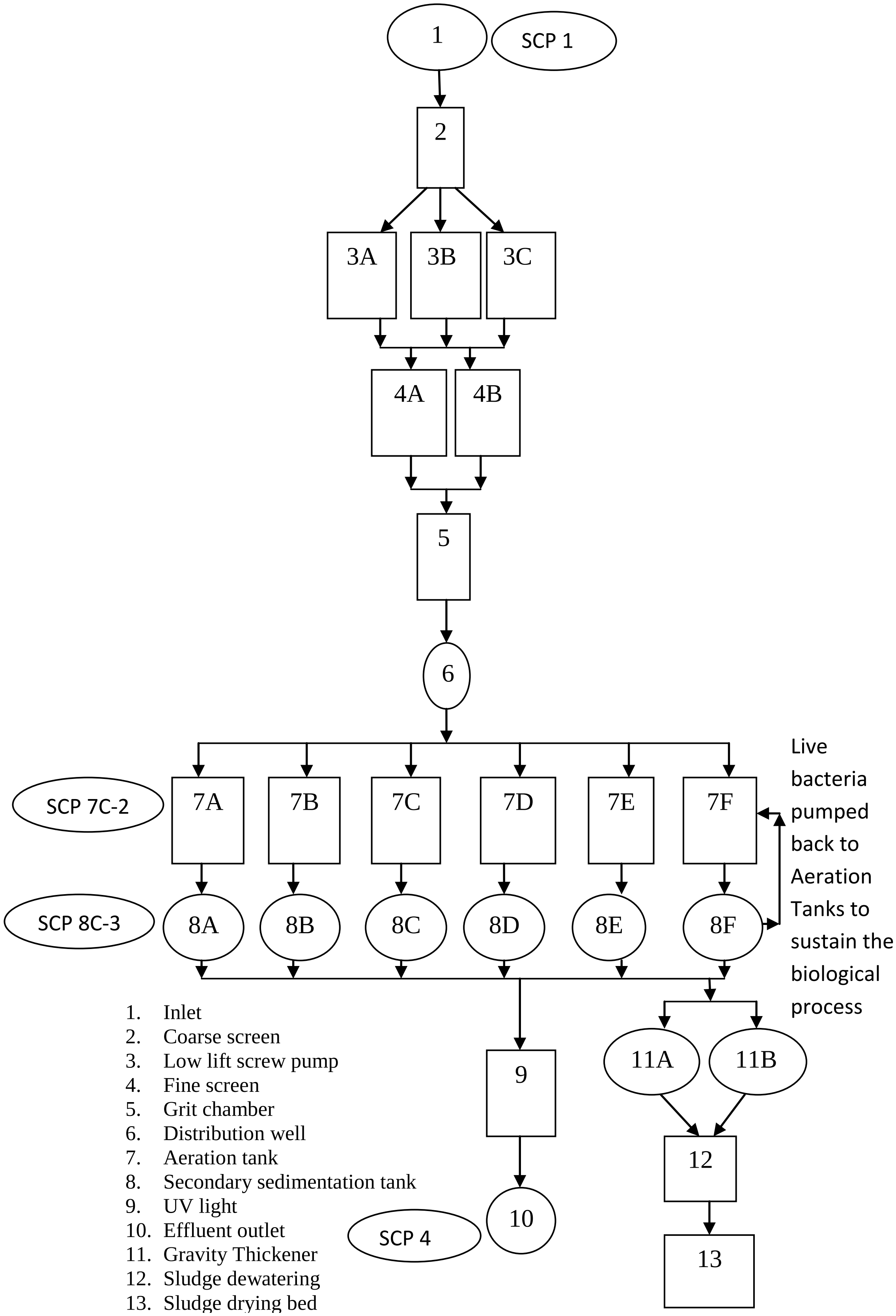


Figure 3.1: Schematic diagram showing sample collection points (SCP) at WSTP Abuja

3.2.4 Statistical Analysis

The removal percent was measured as:

$$\text{Removal }\% = \frac{100(C_o - C_e)}{C_o} \tag{3.1}$$

Where (C_o) and (C_e) are the initial and final concentrations (mg/l) of the wastewater samples respectively.

3.2.5 Sample Analysis

(A) Chemical oxygen demand (COD)

<u>(Closed Reflux Method)</u>

<u>Procedure</u>

1. Suspend the bottom sediment in the reaction cell by swirling.
2. Pipette 3 ml of the pretreated sample and carefully allow it to run from the pipette down the side of the tilted reaction cell onto the reagent.
3. Tightly attach the screw cap to cell. The cell must be held only by the screw cap
4. Vigorously mix the content of the cell
5. Heat the cell at 148˚C in the preheated thermo reactor for 120 min.
6. Remove the hot cell from the thermo reactor and allow to cool in a test tube rack.
7. Wait for 10 min. swirl the cell and return to the rack for complete cooling to room Temperature (cooling time at least 30 min.)
8. Measure in the photometer.

(B) Biochemical oxygen demand (BOD_5)

(Respirometric)

1. Measure 600 ml of sample into a beaker and place on a magnetic stirrer, saturate for 10 minutes.

2. Rinse the measuring bottle and BOD bottles with the sample to be analyzed.

3. Measure 164 ml or 432 ml of sample, 164 ml for concentrated samples such as inlet and 432 ml for less concentrated samples like effluent, upstream.

4. Pour the measured sample into the BOD bottle with the help of a funnel.

5. Insert the quiver and add 2 full, whole, round pellets of NaOH.

6. Cover with the read out cover.

7. Press and hold down S and M button together on the cover until the reading changes to 0, 0

8. Incubate at 20˚C for 5 days.

(C) Dissolved oxygen (DO)

(Titrimetric)

Procedure

1. Rinse the Oxygen reaction bottle several times with the sample, taking care that the mixing element does not fall out.
2. Fill bubble-free sample into the oxygen reaction bottle overflowing.
3. Add 5 drops of O_2-1 (Magnessium)
4. Add 5 drops of O_2-2 (Sodium hydroxide)
5. Close the bottle bubble-free with the ground stopper.

6. Mix thoroughly for 10 seconds
7. Leave the closed oxygen reaction bottle to stand for 1 minutes at constant temperature (a precipitate may be formed)
8. Add 10 drops of O_2-3 (H_2SO_4)
9. Close the bottle bubbles free with the ground glass stopper
10. Mix thoroughly
11. Rinse the test vessel several times with the samples reagent mixture.
12. Inject 5 ml of sample reagent mixture into a vessel with the syringe.
13. Add 1 drop of O_2-4 and swirl (Starch)
14. Depending on the oxygen content, the solution turns violet to blue in colour.
15. Place the titration pipette loosely on the open reagent bottle O_2-5. (Na_2SO_3)
16. Slowly withdraw the piston of the titration pipette from its lowest position until the lower edge of the lack piston seal is level with the zero mark of the scale.
17. Remove the titration pipette and briefly wipe the drop of the dropping tube.
18. Then slowly add the titration solution drop wise to the sample while swirling until the sample becomes entirely coloureless, shortly before the colour changes, wait a few seconds after add each drop.
19. Read off the result in mg/l from the scale of the titration pipette at the lower edge of the back piston.

(D) Total suspended solids

<u>(Gravimetric)</u>

<u>Procedure</u>

1. Pre-treat filter paper by flooding it with distilled water and heating at 105˚C for 2 hours in the oven

2. Cool in the desicator and weigh

3. Record it as B

4. Measure 100 ml of sample, mix it and flood with the mixed sample

5. Suck the liquid out by using suction pump

6. Dry the filter paper and residue in the oven at 105˚C for 2 hrs

7. Bring it out after 2 hrs, cool and weigh

8. Record as A

<u>CALCULATION</u>

The total suspended solids are expressed as:

Total suspended solids (TSS) = $\frac{A-B}{Sample\ Volume} X\ 1000$ mg/l

(E) Total coliform count (TCC)

<u>Pour Plate Method</u>

<u>Procedures</u>

1. Dissolve 2.4g of Mackonkey agar in 50 ml of distilled water

2. Autoclave at 121˚C for 15 minutes at 15 psi

3. Into a clean sterilized petri dish, pipette 1 ml of innoculum (sample).

4. Pour over it molten agar (Mackonkey) prepared. Then swirl for mixture (enough

quantity to cover the bottom of the petric dish)

5. Allow it to solidify

6. Invert the petri dish

7. Incubate at 37˚C for 24 hrs

8. After 24 hrs count all the pinkish red, distinct colony on the plate. Coliform appear pinkish red on Mackonkey agar.

(F) Faecal Coliform (FC)

Standard Fermentation Technique

Procedure

1. Check direction for preparation on the manufacturers manual (EC Broth).
2. Autoclave
3. Use 15 bottles for serial dilution of the sample i.e five bottles per dilution (1.0, 0.1 and 0.01).
4. Label each of the bottles accordingly.
5. Introduce 10ml of the broth prepared into the fermentation bottles (Maccarney bottles).
6. Introduce 1ml of sample into the fermentation bottles containing the 10ml broth
7. Insert the Dorham tube (inverted) into the medium
8. Cork back the bottle
9. Put into the incubator at 44.5˚C for 48hrs
10. Check for acid and gas production. If there is fermentation- Positive, if there is no Fermentation- Negative. For fermentation colour change to yellowish

11. Check No of positive bottles (fermented)

12. Check against the standard chart to determine the result. If all bottles are positive >1600.

(G) Ammonium as N (NH_4-N)

(Colorimetric)

Procedure

1. Pipette 5 ml of sample into a clean test tube
2. Add 5 drops of NH_4^- - 1 Shake
3. Add 5 drops of NH_4^- - 2 Shake
4. Add 5 drops of NH_4^- - 3 Shake
5. Leave to stand for 10 minutes
6. Check the colour change against the colour map

(H) Nitrate as N (NO_3-N)

Colorimetric

Procedure

1. Pipette 5 ml of sample into a test tube
2. Add 6 drops of NO_3^- - 1 Shake
3. Add 1 disc of NO_3^- - 2
4. Shake till it dissolves completely
5. Leave for 10 minutes
6. Then check against the colour map

(I) Nitrite as N (NO_2-N)

Colorimetric

Procedure

1. Pipette 5 ml of sample into a test tube
2. Add 6 drops of NO_3^--1 Shake
3. Shake for 2 minutes
4. Check against the colour map

(J) pH

Electrometric method

Procedure

1. Measure 100 ml of sample in a beaker
2. Rinse the electrode with the sample
3. Insert the electrode into the beaker
4. Read off the pH from the pH meter

(K) Chloride (CL^-)

Photometric Method

Procedure

1. Pipette 1 ml of the sample into a test tube
2. Add 2.5 ml of reagent Cl^--1 with pipette and mix
3. Add 0.5 ml of reagent Cl^--2 with pipette and mix
4. Leave to stand for 1 minute (reaction time)

5. Fill the sample into a cell (photo cell)

6. Measure in the photometer

*pH of the measuring solution must be approximately 1, adjust if necessary with sulphuric acid.

(L) Sulfate (SO_4^-)

Photometric Method

Procedure

1. Pipette 1 ml of sample into a reaction cell and mix, add 1 level green micro spoon of reagent Barium chloride, close the cell tightly and shake vigorously until the reagent is completely dissolved.

2. Leave to stand for exactly 2 minutes (reaction time)

3. Measure in the photometer

*pH of the sample must be within the range of 2 - 10)

*pH of the measuring solution must be within the range of 1 - 2

(M) Electrical Conductivity (EC)

Electrometric method

Procedure

5. Measure 100 ml of sample in a beaker

6. Rinse the electrode with the sample

7. Insert the electrode into the beaker

8. Read off the EC from the EC meter

CHAPTER FOUR

RESULTS AND DISCUSSIONS

The discussions are presented in threefold: first, it looked at the performance of the WUPA sewage treatment plant (WSTP) in terms of biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), total suspended solids (TSS), dissolved oxygen (DO), total coliform Count (TCC), faecal coliform (FC), ammonium as N (NH_4-N), nitrate as N (NO_3-N), nitrite as N (NO_2-N), sulfate (SO_4^-), chloride (CL^-), pH and electrical conductivity (EC) removal/increase percentage. Secondly, it studied the effect of the treatment plant on the effluent quality of the receiving water body in terms of the aforementioned parameters and lastly, the maintenance plan for the WUPA sewage treatment plant was developed. In all threefold, the result were presented and interpreted with theoretical and practical knowledge that what is the performance of WSTP in terms of the physicochemical and bacteriological parameters.

4.1.1 Biochemical oxygen demand (BOD_5) removal

Table 4.1 Average biochemical oxygen demand (BOD_5) characteristics of the treatment plant from May-September, 2015

Period		**Influent (mg/l)**	**Effluent (mg/l)**	**Removal efficiency (%)**	**On river WUPA**	
					10m upstream (mg/l)	**10m downstream (mg/l)**
May-Sept., 2015	Mean	126.08	9.33	92.42	13.50	12.50
	S.D	52.22	2.61	4.04	5.33	5.04
	SKEW	0.12	1.46	1.63	0.81	0.85
W.H.O standards		30				
NESREA Effluent limits		30				

The influent biochemical oxygen demand (BOD_5) of the treatment plant had an average value of 126.08 mg/l (Table 4.1), and standard deviation of 52.22 which signifies the variation in the influent quality and skewness coefficient of 0.12. The effluent after UV had an average concentration of 9.33 mg/l and standard deviation of 2.61, having skewness coefficient of 1.46, details of which is shown in Table A.1. Moreover, the treatment plant was observed to have achieved 92% removal efficiency as opined by Devendra and Mahesh, (2014), the removal of BOD is by a biological process, such as the suspended growth treatment process. This biological process is an aerobic process and takes place in the aeration tank, in which the wastewater is aerated with oxygen. The result of analysis from the treatment plant's laboratory was obtained to study the characteristic of the influent and effluent throughout the year detail of which is as shown below:

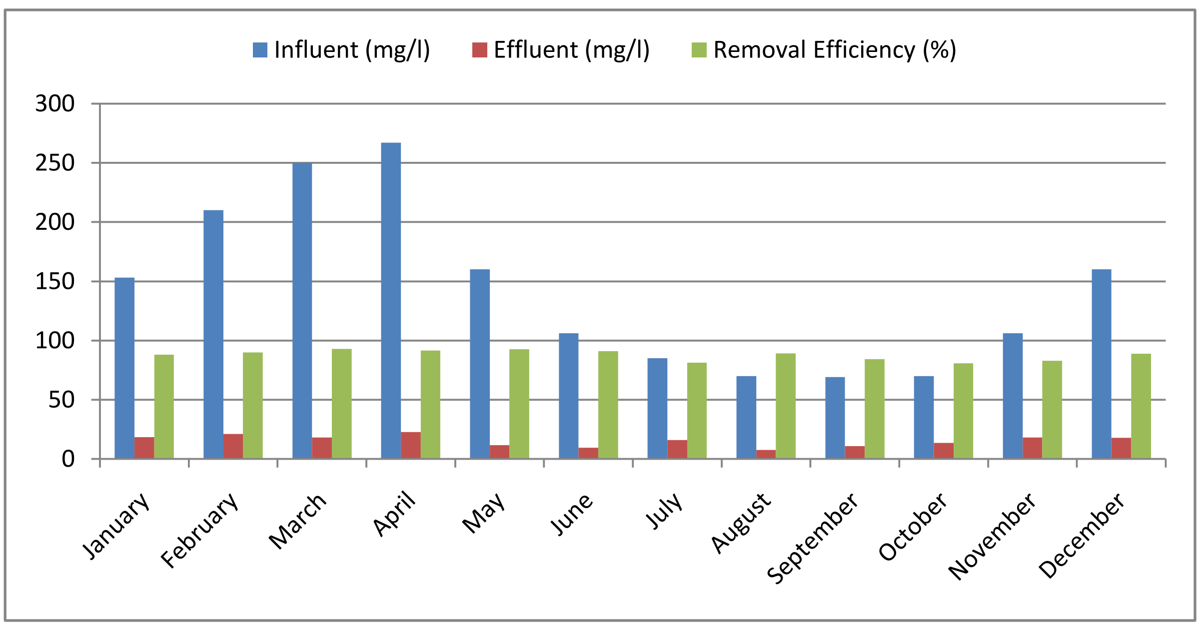


Fig. 4.1 Average biochemical oxygen demand (BOD_5) characteristics of the treatment plant from January-December, 2014

March and April indicated high influent concentration values of 250 mg/l and 267 mg/l respectively and the percentage removal efficiency for the annual data ranged from 80% -

92% (Table B.1). It can be concluded from the chart above that the concentration of the influent is influence by the seasonal variation in Nigeria as opined by Longe and Ogundipe (2010).

Moreover, BOD_5 removal is indicative of the efficiency of biological treatment processes (Sincero and Sincero, 1996). Special consideration has been given in the current study to the organic content, characterized by BOD_5, COD and the COD/BOD_5 ratio.

From Table 1.2, the design BOD_5 loading for the treatment plant was 42,000 kg/d, hence the design BOD_5 for the influent is 320 mg/l and Table 4.1 revealed an averaged BOD_5 of 126.08 mg/l equivalent to 4,961.25 kg/d BOD_5 loading, as the system is presently operating at 30% design capacity.

4.1.2 Chemical oxygen demand (COD) removal

The chemical oxygen demand (COD) concentrations for the treatment plant as observed during the study are as shown in Table 4.2 below.

Table 4.2 Average chemical oxygen demand (COD) characteristics of the treatment plant from May-September, 2015

Period		**Influent (mg/l)**	**Effluent (mg/l)**	**Removal efficiency (%)**	**On river WUPA**	
					10m upstream (mg/l)	**10m downstream (mg/l)**
May-Sept., 2015	Mean	283.58	46.41	83.18	42.25	40.66
	S.D	93.53	12.44	2.56	12.01	13.59
	SKEW	0.09	-0.14	-0.50	0.13	0.08
W.H.O standards		100				
NESREA Effluent limits		80				

The influent COD concentration of the wastewater had an average value of 283.58 mg/l with standard deviation of 93.53 and skewness of 0.09 (Table 4.2). After undergoing treatment from the aeration tank to the sedimentation basin, the effluent COD was observed to have an average concentration of 46.41 mg/l and standard deviation of 12.44 signifying that there was a wide range of variation in the effluent concentration as the skewness coefficient of -0.14 was determined.

An average of 83% removal efficiency was achieved after the treatment, indicating a high efficiency performance in terms of COD for the treatment plants. The result contained in (Table 4.2 and Table A.2) relates to the COD characteristics of the treatment plant for the period of May–September, 2015 during which this research was conducted.

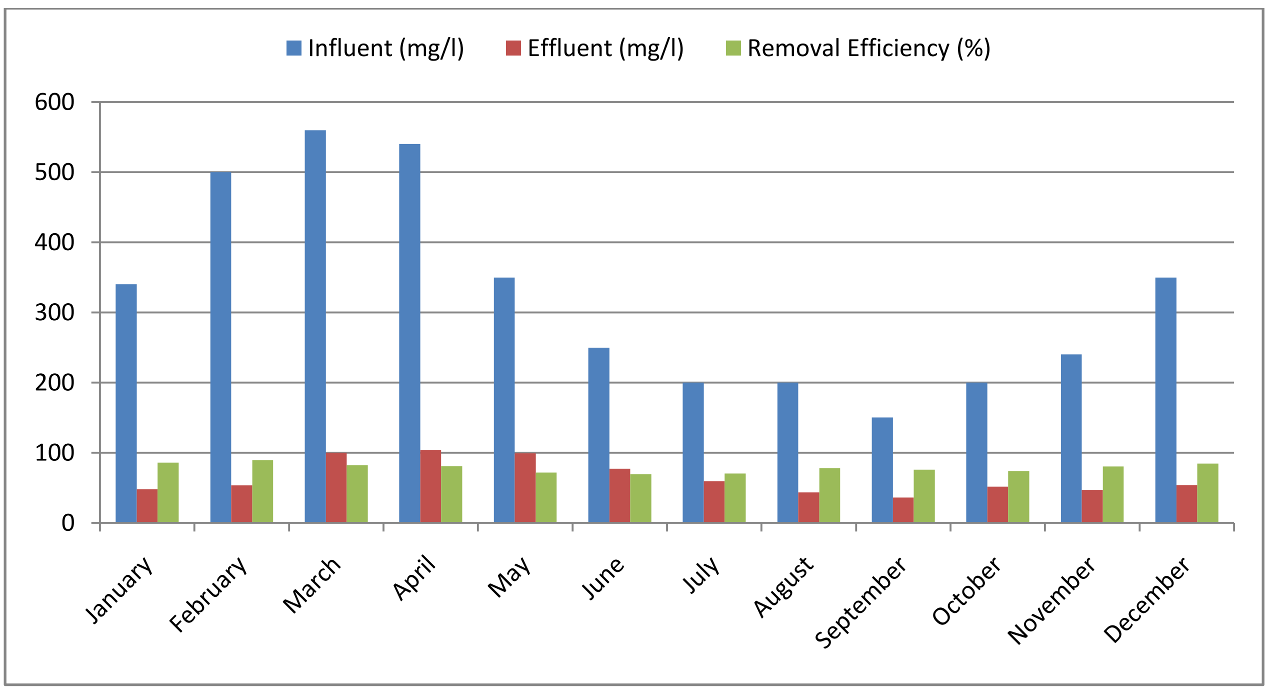


Fig. 4.2 COD relationship between the influent, effluent and removal efficiency for the treatment plant from January-December, 2014

In consideration of the effect for the seasonal variation, an annual result (January-December, 201) from the treatment plants laboratory was collected and evaluated as shown above. The chart indicated that maximum influent concentration of 500 mg/l and 560 mg/l were observed in February and March respectively while April had 540 mg/l concentration but minimum concentration of 200 mg/l were observed in April and July, 2014. Moreover, maximum removal efficiency of 89% was achieved.

For an activated sludge treatment process the biodegradation/digestion of the wastewater takes place at the aeration basin with sludge sedimentation at the sedimentation tank (Ukpong, 2013). Also Table A.2 revealed that there was a large fluctuation in the COD of the influent giving rise to a large fluctuation in the COD removal efficiency of the treatment plant as a result of high dilution of the wastewater being treated (Ravi *et al*., 2010). The fluctuation in the concentration of BOD_5 and COD is indicative of the need for the influent wastewater to be pretreated before been discharged into the treatment plant to achieve the required removal efficiency (Sushil, 2008).

The COD/BOD_5 ratio is also an important factor which determines the biodegradability of domestic sewage and industrial wastewater. The COD/BOD_5 ratio varied from 1.7 to 2.4 for raw domestic sewage; however it varied widely for industrial wastewater. The ratio of COD to BOD_5 of wastewater and its indication for biodegradability is shown in Table 4.3

Table 4.3 Ratios of wastewater COD to BOD_5 and their indication

COD/BOD_5 ratio	**Indication**
Less than 2.5	Biodegradable fraction is high
Between 2.5 – 4.0	The inert (non-biodegradable) fraction is not high
More than 4.0	The inert (non-biodegradable) fraction is high

Source: Adopted from Sperling and Chernicharo, 2005

It is noticed from Table 4.4 that BOD_5 and COD removal were 90%, 95% and 82%, 87% when the influent COD/BOD_5 ratio were 2.09 and 1.98 respectively, which was less than 2.5. The high BOD_5 removal was achieved due to the presence of large fraction of biodegradable matter in the wastewater. Less BOD_5 removal was achieved when the COD/BOD_5 ratio of influent was more than 2.5. This was due to less biodegradable fraction; hence COD was determined to ascertain the extent of the non-biodegradable inorganic matter in the wastewater.

Table 4.4 Influence of COD to BOD_5 ratio on BOD_5 removal efficiency of the treatment plant

Period	**BOD_5 Influent (mg/l**	**Effluent (mg/l)**	**BOD_5 removal %**	**COD Influent (mg/l)**	**Effluent (mg/l)**	**COD removal %**	**Influent COD/BOD ratio**	**Effluent COD/BOD ratio**
Week 1	160.00	16.00	90.00	334.00	60.00	82.04	2.09	3.75
Week 2	170.00	12.00	92.94	369.00	63.00	82.93	2.17	5.25
Week 3	180.00	10.00	94.44	400.00	60.00	85.00	2.22	6.00
Week 4	180.00	8.00	99.56	380.00	55.00	85.53	2.11	6.88
Week 5	200.00	10.00	95.00	395.00	50.00	87.34	1.98	5.00
Week 6	160.00	8.00	99.50	350.00	55.00	84.29	2.19	6.88
Week 7	75.00	8.00	89.33	210.00	45.00	78.57	2.80	5.63
Week 8	80.00	10.00	87.50	200.00	42.00	79.00	2.50	4.20
Week 9	78.00	6.00	92.30	195.00	35.00	82.05	2.50	5.83
Week10	85.00	8.00	90.58	200.00	30.00	85.00	2.35	3.75
Week11	70.00	8.00	88.57	190.00	32.00	83.15	2.71	4.00
Week12	75.00	8.00	89.33	180.00	30.00	83.33	2.40	3.75
Mean	126.08	9.33	92.42	283.58	46.41	83.18	2.25	4.97

4.1.3 Total suspended solid (TSS) removal

The characteristics of the treatment plant in terms of total suspended solid concentration and removal efficiency are as contained in the table below;

Table 4.5 Average total suspended solid (TSS) characteristics of the treatment plant from May-September, 2015

Period		**Influent (mg/l)**	**Effluent (mg/l)**	**Removal efficiency (%)**	**On river WUPA**	
					10m upstream (mg/l)	**10m downstream (mg/l)**
May-Sept., 2015	Mean	146.91	12.58	89.49	37.00	29.33
	S.D	43.09	4.08	5.13	13.95	13.04
	SKEW	0.60	1.54	-2.46	0.18	0.50
W.H.O standards		30				
NESREA Effluent limits		30				

A wide range of concentration was observed for TSS at the influent to the treatment plant with an average value of 146.91 mg/l, standard deviation of 43.09 and skewness coefficient 0.60. The treatment process was able to reduce these concentrations to an average of 12.58 mg/l at the effluent outlet with marginal deviation of 4.08 and skewness coefficient of 1.54. An assessment of the treatment plant in terms of efficiency indicated an average of 89% removal efficiency. This high efficiency is indicating the biodegradability of the wastewater by microorganisms (Saeid and Hosen, 2014).

The chart below (Fig. 4.5) reveals the characteristics of the treatment plants in terms of its influent, effluent concentrations and removal efficiency for one year period (Table B.5). The month of March recorded the maximum influent concentration of 299 mg/l after which removal efficiency of 95% was achieved. April and February had concentrations of 240 mg/l and 209 mg/l with removal efficiencies of 92% and 91% respectively while January and December recorded influent concentrations of 126 mg/l and removal efficiencies of 88% and 86% respectively.

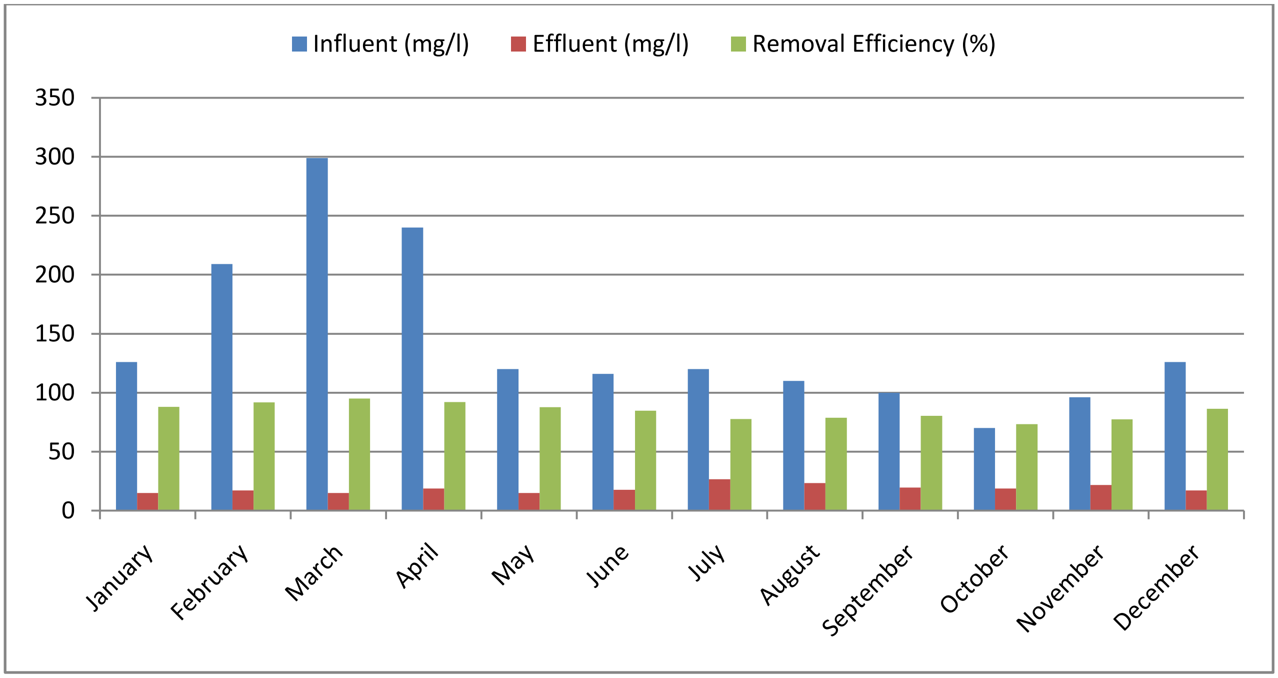


Fig. 4.5 TSS relationship between the influent, effluent and removal efficiency for the Treatment plant from January-December, 2014

October had the least influent concentration of 70.00 mg/l and effluent of 18.70 mg/l, the removal efficiency achieved was 73%.

4.1.4 Dissolved oxygen (DO) increase

Table 4.6 Average dissolved oxygen (DO) characteristics of the treatment plant from May-September, 2015

Period		**Influent (mg/l)**	**Effluent (mg/l)**	**Increase in efficiency (%)**	**On river WUPA**	
					10m upstream (mg/l)	**10m downstream (mg/l)**
May-Sept., 2015	Mean	4.05	7.44	46.47	6.54	7.24
	S.D	1.43	0.75	15.03	0.75	1.01
	SKEW	-0.18	0.08	0.41	0.08	0.20
W.H.O standards		7-10				
NESREA Effluent limits		7-10				

The average influent dissolve oxygen concentration for the treatment plant during this research period was 4.05 mg/l with standard deviation of 1.43 and skewness coefficient of -

0.18, the effluent concentration was observed to be 7.44 mg/l which is an average of 46% increase in the dissolved oxygen concentration of the effluent water. This effluent concentration deviated from the mean concentration by 0.75 and skewness coefficient of 0.08 (Table 4.6).

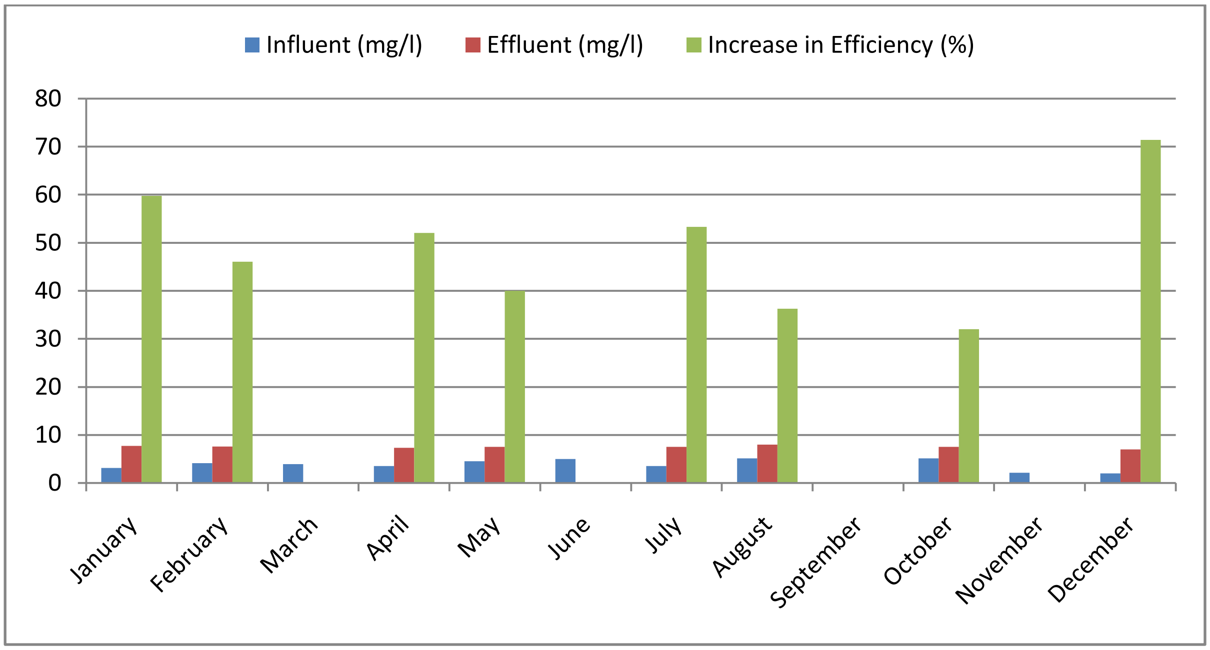


Fig. 4.6 DO relationship between the influent, effluent and increase in efficiency for the treatment plant from January-December, 2014

Fig 4.6 above shows an annual trend in the dissolved oxygen concentration of the treatment plant. The month of December recorded the highest increase in efficiency of 71% with influent and effluent concentrations of 2.00 mg/l and 7.00 mg/l respectively while October had the least increase in efficiency of 32% with influent and effluent concentrations of 5.10 mg/l and 7.50 mg/l respectively (Table B.6).

Nkwocha *et al.*, (2013) opined that DO concentration greater than 5.00 mg/l gives a better quality effluent. The microscopic investigation showed a tendency toward poorer sludge properties at a concentration of 2.00 mg/l compared to 4.00 mg/l (Prachi and Sameer, 2014).

Aeration of activated sludge is very costly and therefore it is desirable to operate the treatment plant at low DO concentration as possible without risking effluent quality (Devendra and Mahesh, 2014) also DO concentration is important because nitrifying bacteria are sensitive to low DO (≤2.0 mg/L) and because aeration is a major factor associated with aerobic wastewater treatment (Grady et al., 1999). Dissolved oxygen concentration can also have a major impact on ammonia removal in activated sludge. Since a DO concentration at or above 2 mg/L has been established as the minimum necessary to prevent inhibition (Parker, 2001).

4.1.5 Nitrate as N (NO_3-N) increase

Table 4.7 Average Nitrate as Nitrogen (NO_3-N) characteristics of the treatment plant from May-September, 2015

Period		**Influent (mg/l)**	**Effluent (mg/l)**	**Increase in efficiency (%)**	**On river WUPA**	
					10m upstream (mg/l)	**10m downstream (mg/l)**
May-Sept., 2015	Mean	1.18	3.40	60.52	1.70	1.47
	S.D	0.25	1.07	18.14	0.57	0.42
	SKEW	-0.48	-0.19	-1.29	0.55	0.78
W.H.O standards		10				
NESREA Effluent limits		20				

The influent wastewater had an average nitrate concentration of 1.18 mg/l, standard deviation of 0.25 and skewness coefficient of -0.48; at the effluent outlet an average concentration of 3.40 mg/l was observed with standard deviation of 1.07 and skewness coefficient of -0.19, this translate to an average increase in efficiency of 60% for the treatment plant (Table 4.7).

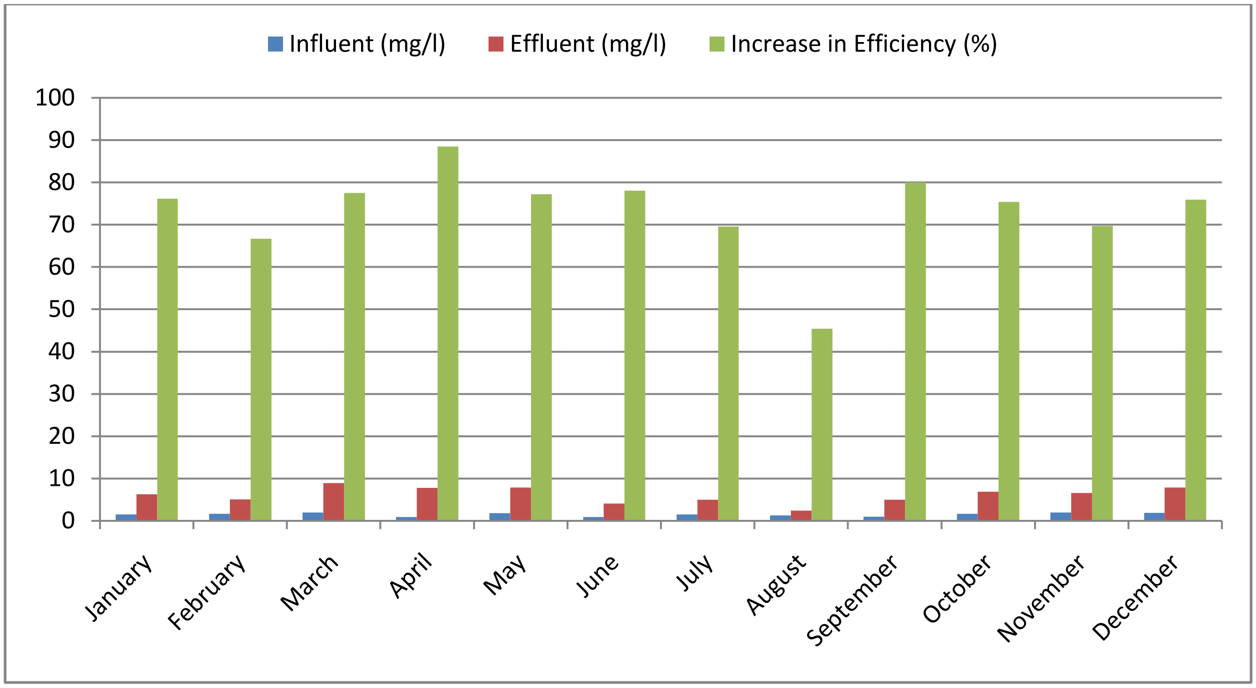


Fig. 4.7 NO_3-N relationship between the influent, effluent and increase in efficiency for the treatment plant from January-December, 2014

Fig. 4.7 above relates to an annual nitrate as nitrogen trend for the treatment plant; the highest increase from 0.9 mg/l to 7.80 mg/l was recorded in the month of April translating to 88% increase in efficiency. The month of September had an influent and effluent concentration of 1.00 mg/l and 5.00 mg/l respectively and 80% increase in efficiency. Moreover, the least increase in efficiency of 45% was observed in August, which recorded 1.31 mg/l and 2.40 mg/l for influent and effluent respectively (Table B.7).

NO_3-N increase indicates that nitrogen is being assimilated for the formation of biomass. Nitrification rates in wastewater depend upon the dissolved oxygen concentration. Many researchers have suggested an optimum DO concentration of 2 to 3 mg/L, together with a BOD of less than 20 mg/L and COD of 30-40 mg/L, needed for nitrification to occur (Hashmi, 2007; Eckenfelder, 1989; Barnes and Bliss, 1983). The increase NO_3-N concentration indicates that concentration of dissolved oxygen was not up to the mark where rapid conversion of NH_3-N occurs.

4.1.6 Nitrite as Nitrogen (NO_2-N) increase

Table 4.8 Average Nitrite as Nitrogen (NO_2-N) characteristics of the treatment plant from May-September, 2015

Period		**Influent (mg/l)**	**Effluent (mg/l)**	**Increase in efficiency (%)**	**On river WUPA**	
					10m upstream (mg/l)	**10m downstream (mg/l)**
May-Sept., 2015	Mean	0.41	0.42	71.55	0.71	0.53
	S.D	0.24	0.34	11.43	0.22	0.26
	SKEW	1.34	0.38	0.18	0.56	0.83
W.H.O standards		2				
NESREA Effluent limits		1				

The influent Nitrite as Nitrogen (NO_2-N) had an average concentration of 0.41 mg/l, standard deviation of 0.24 and skewness coefficient of 1.34 while the effluent had an average concentration of 0.42 mg/l, standard deviation of 0.34 and skewness coefficient of 0.38, an average of 72% increase in efficiency was achieved (Table 4.8).

Fig. 4.8 below shows the relationship between the nitrite concentration for the influent, effluent and removal efficiency for the treatment plant within one year period, while during the research period, an increase in efficiency was observed, for this data removal efficiency was actually observed for the same treatment plant (Table B.8).

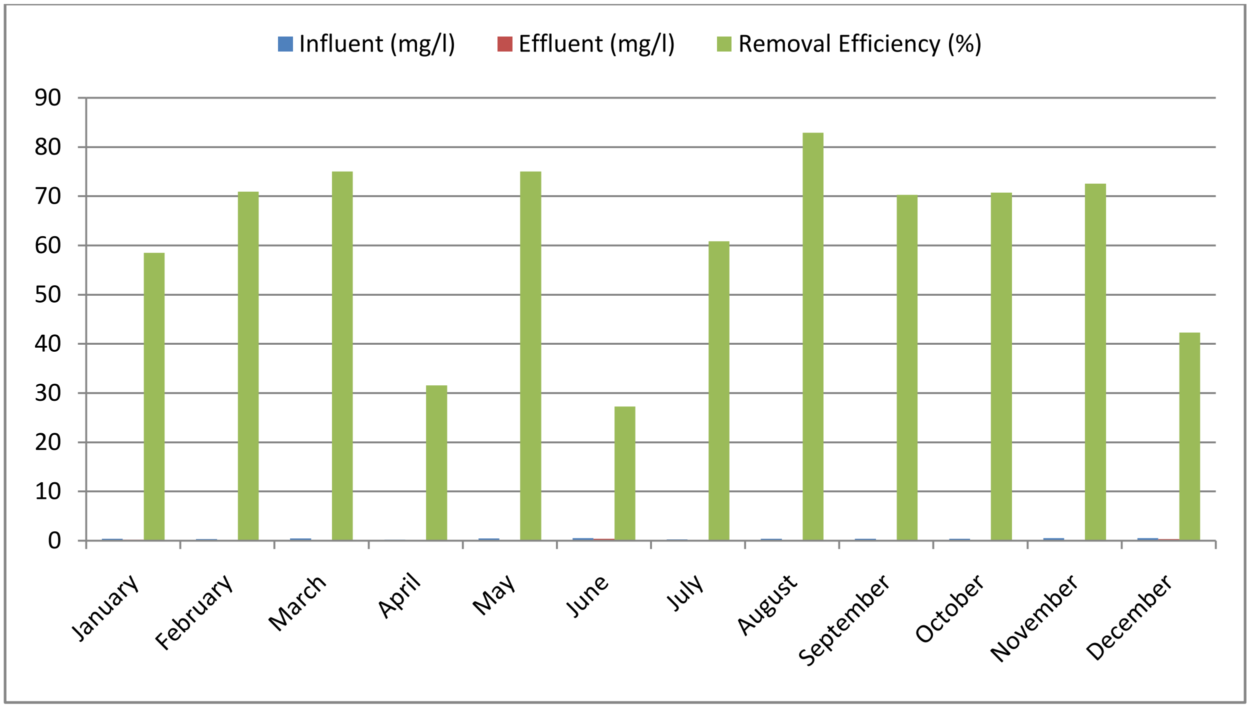


Fig. 4.8 NO_2-N relationship between the influent, effluent and removal efficiency for the treatment plant from January-December, 2014

The figure above indicates that August record concentration of 0.41 mg/l and 0.07 mg/l for influent and effluent respectively, with an average removal efficiency of 83% while March and May both had influent and effluent concentrations of 0.44 mg/l and 0.11 mg/l respectively, attaining removal efficiencies of 75%. The least removal efficiency 27% was achieved in June.

4.1.7 Ammonium as Nitrogen (NH_4-N) removal

Table 4.9 Average Ammonium as Nitrogen (NH_4-N) characteristics of the treatment plant from May-September, 2015

Period		**Influent (mg/l)**	**Effluent (mg/l)**	**Removal efficiency (%)**	**On river WUPA**	
					10m upstream (mg/l)	**10m downstream (mg/l)**
May-Sept., 2015	Mean	4.12	0.79	58.29	1.35	0.88
	S.D	3.15	0.54	39.49	0.29	0.07
	SKEW	1.47	0.03	-0.13	-0.28	0.19
W.H.O standards		-				
NESREA Effluent limits		-				

The table above indicates that the ammonium influent concentration for the treatment plant is 4.12 mg/l, standard deviation of 3.15 showing that a wide range of value existed for the concentration with skewness coefficient of 1.47. The effluent had mean concentration of 0.79 mg/l, standard deviation of 0.54 and skewness coefficient of 0.03 with removal efficiency of 58%.

Fig. 4.9 below shows the annual relationship between the influent, effluent and percentage increase in efficiency for the treatment plant. While during the period of conducting this research work, a removal in efficiency was observed but from this data obtained from quality control manager's office, an increase in efficiency was observed. The month of June recorded 96% increase in efficiency, in which the influent increases in concentration from 0.52 mg/l to 12.50 mg/l; July had 88% increases in efficiency with influent and effluent concentration of 0.49 mg/l and 4.10 mg/l respectively. Furthermore, 14% increase in efficiency was observed in September (Table B.9).

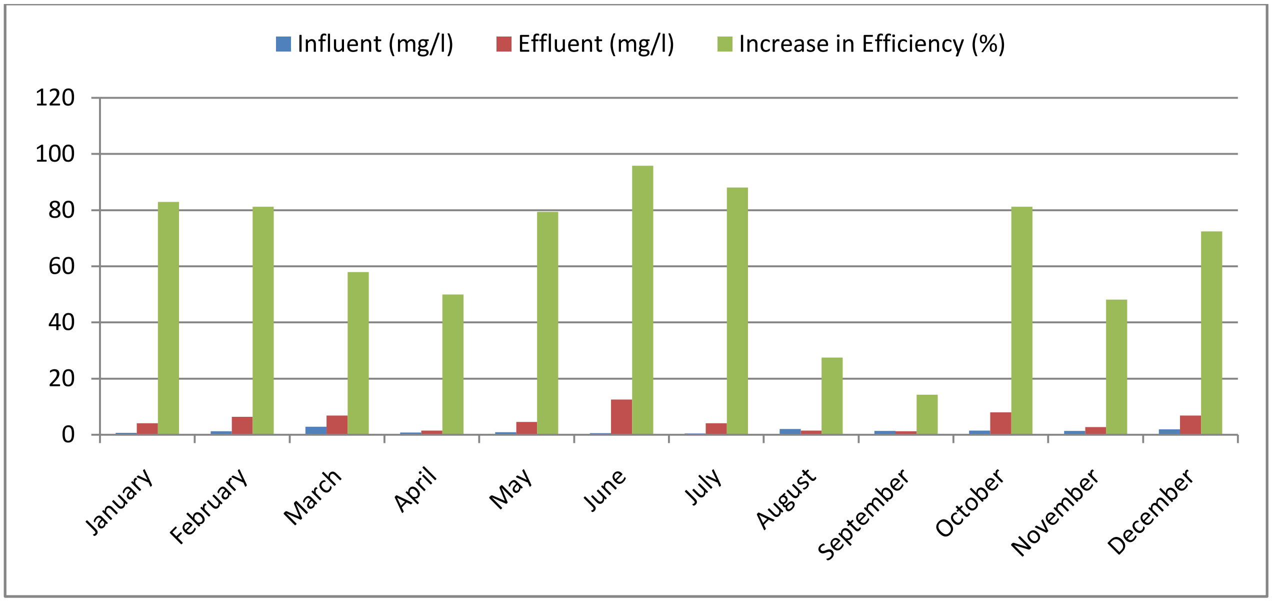


Fig. 4.9 NH_4-N relationship between the influent, effluent and increase in efficiency for the treatment plant from January-December, 2014

4.1.8 Chloride (CL^-) removal

Table 4.10 Average Chloride (CL^-) characteristics of the treatment plant from May-September, 2015

Period		**Influent (mg/l)**	**Effluent (mg/l)**	**Removal efficiency (%)**	**On river WUPA**	
					10m upstream (mg/l)	**10m downstream (mg/l)**
May-Sept., 2015	Mean	24.88	20.18	19.02	17.20	17.53
	S.D	6.27	4.45	8.95	2.46	2.20
	SKEW	1.22	0.26	-0.37	0.44	0.49
W.H.O standards		250				
NESREA Effluent limits		600				

Table 4.10 above contained the average influent, effluent and removal efficiency in terms of chloride characteristics of the treatment plant; an average concentration of 24.88 mg/l was observed at the influent which had a standard deviation of 6.27 and skewness coefficient of 1.22. The average effluent concentration of 20.18 mg/l and standard deviation of 4.45 was

recorded at the effluent outlet having skewness coefficient of 0.26 and removal efficiency of 19% was achieved for the treatment plant.

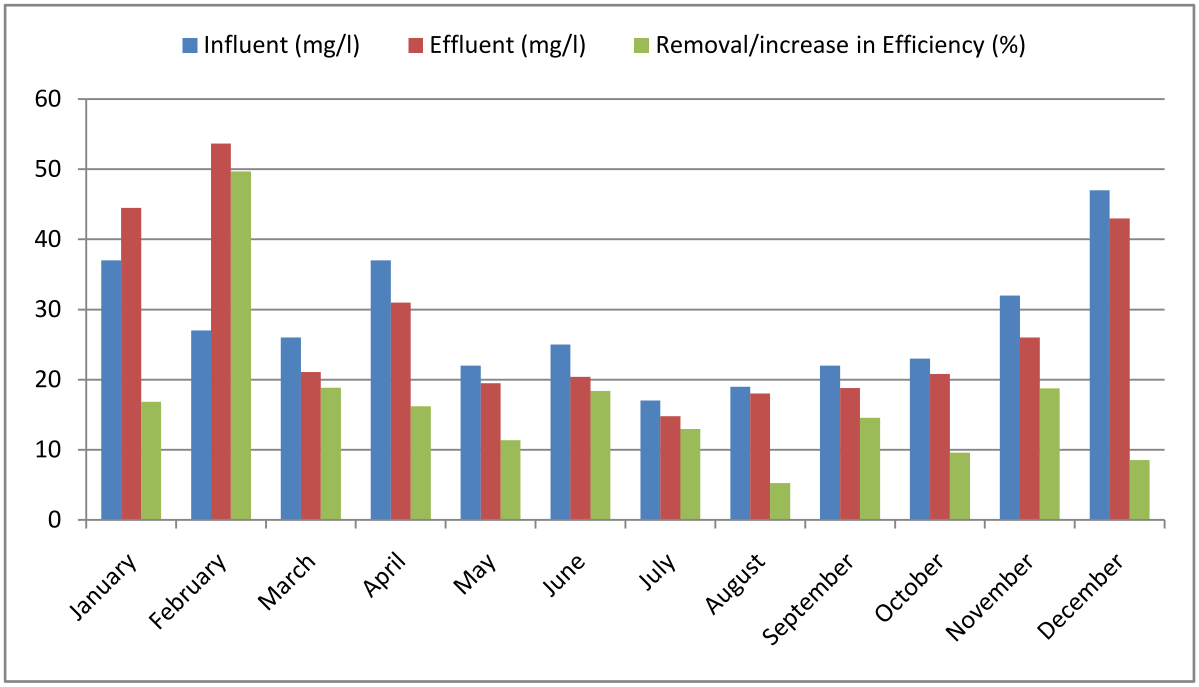


Fig. 4.10 CL^- relationship between the influent, effluent and removal efficiency for the treatment plant from January-December, 2014

The figure above reveals the tread in the characteristics of the influent, effluent and removal efficiency for one year period. The month of January and February recorded influent concentration of 37.00 mg/l and 27.00 mg/l respectively while the effluent had concentration of 44.50 mg/l and 53.67 mg/l respectively which represent an increase in concentration of 17% and 50% respectively. Except for these months, the remaining months of the year recorded decrease in the chloride concentration which indicated removal in efficiency. November and December observed removal efficiency of 19% each while August had the least removal efficiency of 5%; between May and September, the removal efficiency ranged between 5% and 18% (Fig. 4.10).

4.1.9 Sulfate (SO_4^-) removal

Table 4.11 Average Sulfate (SO_4^-) characteristics of the treatment plant from May-September, 2015

Period		Influent (mg/l)	Effluent (mg/l)	Removal efficiency (%)	On river WUPA	
					10m upstream (mg/l)	10m downstream (mg/l)
May-Sept., 2015	Mean	59.91	42.08	30.21	47.75	43.75
	S.D	10.46	10.79	11.26	11.09	9.59
	SKEW	0.75	-0.02	-0.30	0.14	0.07
W.H.O standards		250				
NESREA Effluent limits		500				

Table 4.11 above contained the influent, effluent and removal efficiency characteristics of the treatment plant in terms of sulfate. An average influent concentration of 59.91 mg/l was observed and a standard deviation of 10.46, representing a wide range of values at the inlet having skewness coefficient of 0.75. The effluent had an average concentration of 42.08 mg/l with standard deviation of 10.79 and skewness coefficient of -0.02, the removal efficiency of 30% was achieved for the treatment plant.

The chart below (Fig. 4.11) shows the behaviors of sulfate concentration in terms of influent, effluent and removal/increase in efficiency for the treatment plant; except for January, February and March which indicates decrease in concentration, the remaining months of the year recorded an increase in concentration at the effluent. September had 24% increases in efficiency which is the maximum efficiency throughout the year in consideration.

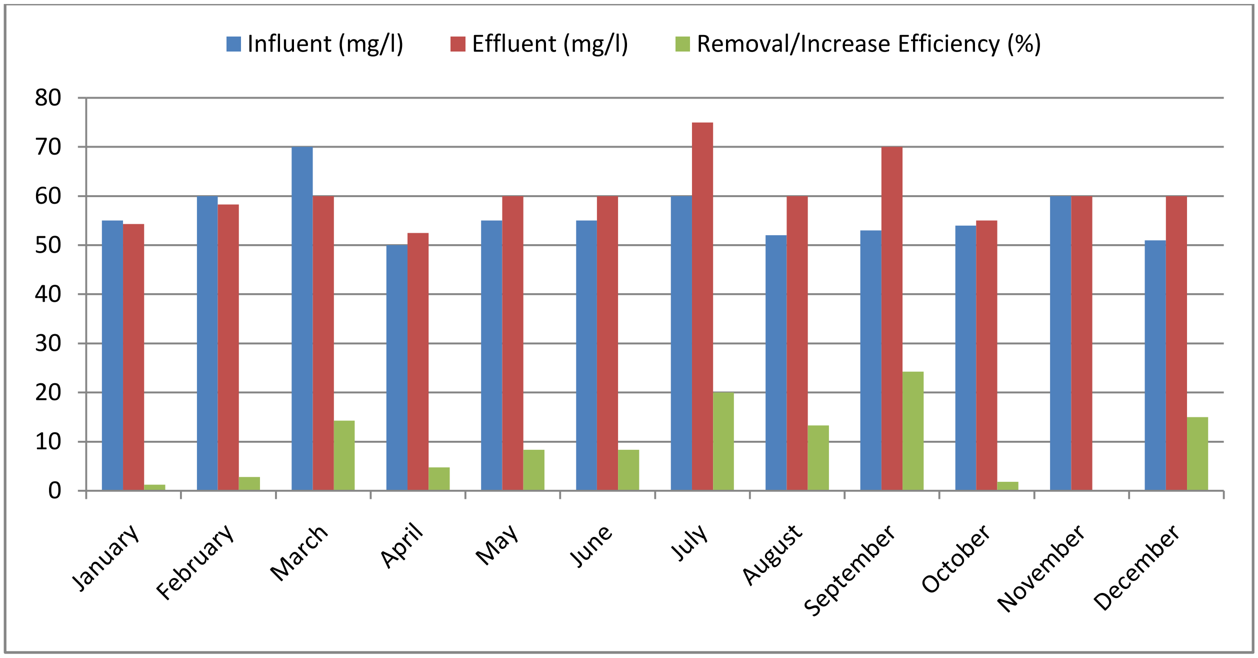


Fig. 4.11 SO_4^- relationship between the influent, effluent and removal/increase in efficiency for the treatment plant from January-December, 2014

4.1.10 Total coliform count (TCC) removal

Table 4.12 Average total coliform count (TCC) characteristics of the treatment plant from May-September, 2015

Period		**Influent (cfu/100ml)**	**Effluent (cfu/100ml)**	**Removal efficiency (%)**	**On river WUPA** 10m upstream (cfu/100ml)	**On river WUPA** 10m downstream (cfu/100ml)
May-Sept., 2015	Mean	1600.00	128.00	91.00	94.00	82.00
	S.D	0.00	63.00	4.00	34.00	37.00
	SKEW	-	1.13	-1.19	0.95	0.61
W.H.O standards		-				
NESREA Effluent limits		-				

The table above (Table 4.12) contained the bacteriological (total coliform count) characteristics of the influent, effluent and removal efficiency for the treatment plant; the influent had 1600 cfu/100ml and the effluent contained 128 cfu/100ml, attaining 91% removal efficiency. This implied that the effluent wastewater discharged into river WUPA contained some percentage of contamination in terms of total coliform count.

Four bacterial species were isolated namely *Escherichia coli*, *Salmonella* spp., *Shigella* spp. and *Streptococcus* spp from the WUPA sewage treatment plant, Abuja (Ugoh, 2013).

4.1.11 Faecal coliform (FC) removal

Table 4.13 Average faecal coliform (FC) characteristics of the treatment plant from May-September, 2015

Period					**On river WUPA**	
		Influent (MPN/100ml)	**Effluent (MPN/100ml)**	**Removal efficiency (%)**	**10m upstream (MPN/100ml)**	**10m downstream (MPN/100ml)**
May-Sept., 2015	Mean	818.00	38.00	93.00	26.00	25.00
	S.D	101.77	45.21	8.90	15.34	16.12
	SKEW	0.22	1.46	-1.59	0.60	0.76
W.H.O standards		-				
NESREA Effluent limits		-				

The faecal coliform characteristic of the treatment plant as at the period of the study is as contained in Table 4.13 above. The influent had an average of 818.00 MPN/100ml, with standard deviation of 101.77 as the values vary widely, but the skewness coefficient of which is 0.22 and 93% removal efficiency was achieved.

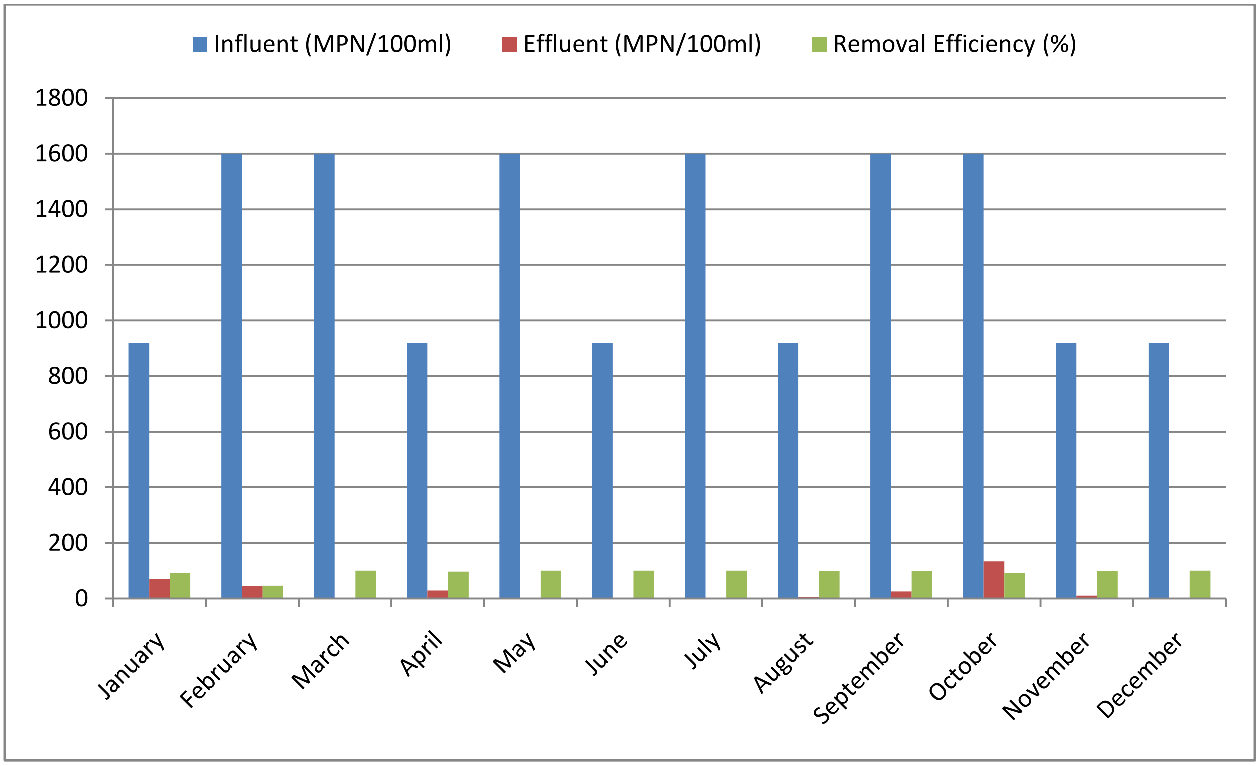


Fig. 4.13 FC relationship between the influent, effluent and removal efficiency for the treatment plant from January-December, 2014

The figure above (Fig. 4.13) is the annual relationship between the influent, effluent and removal efficiency for the treatment plant; an influent average value of 1600 MPN/100ml was observed for the months of February, March, May, July, September and October, with removal efficiency of 46%, 100%, 100%, 100%, 98% and 92% respectively (table B.13).

As opined by Doughari *et al*. (2007), the bacteria isolates from the water belong to the genera of potential pathogenic bacteria, and the microorganisms isolated were *Escherichia coli*, *Streptococcus* spp., *Salmonella* spp. and *Shigella* spp. The isolation of these organisms is of great health concern because this domestic wastewater was collected at the point of discharge into a nearby river, which may not only serve as a source of drinking water to the immediate community but also as a source of food (fishing). *Escherichia coli, Salmonella* spp. and *Shigella* spp. are associated with water borne diseases and reports from available health outposts in the areas in which this study was carried out revealed typhoid fever, dysentery, cholera and hepatitis to be the most prevalent.

4.1.12 pH

Table 4.14 Average pH characteristics of the treatment plant from May-September, 2015

Period		**Influent**	**Effluent**	**Percentage change in Efficiency (%)**	**On river WUPA**	
					10m upstream	**10m downstream**
May-Sept., 2015	Mean	7.26	7.04	3.10	7.22	7.16
	S.D	0.11	0.12	2.81	0.10	0.04
	SKEW	0.63	-0.08	0.52	-1.08	0.99
W.H.O standards		6.5-8.5				
NESREA Effluent limits		6.9				

For biological treatment processes like the activated sludge process in consideration, the pH of the influent is very important which influence the performance of the process. From table 4.14 above, the influent had an average pH value of 7.26 which is within the region of neutrality with standard deviation of 0.11 indicating that the variation in pH was within limit having skewness coefficient of 0.63.

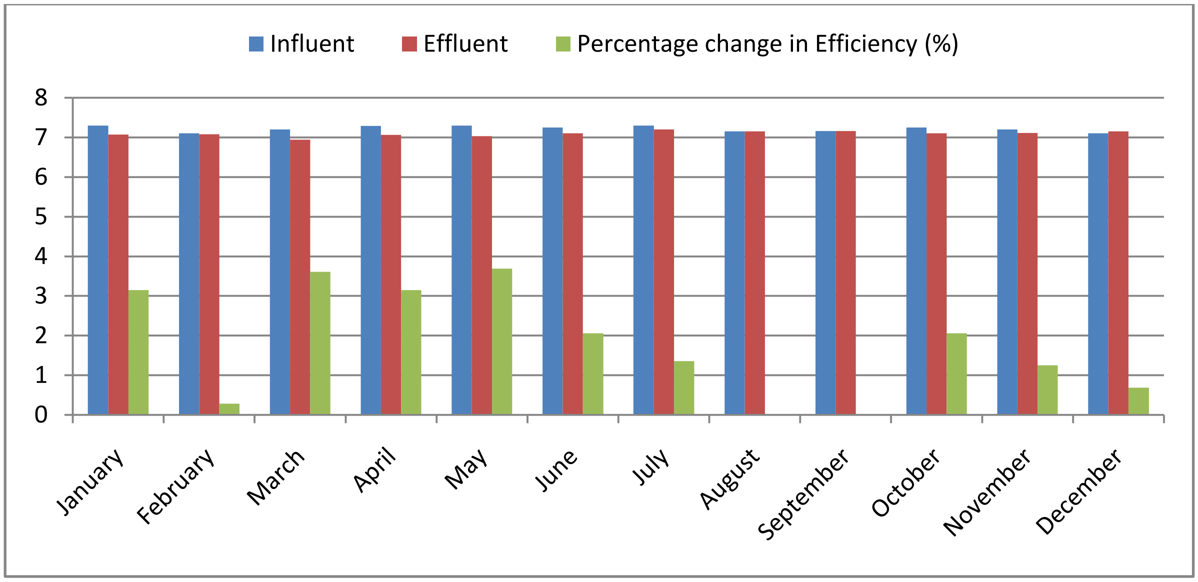


Fig. 4.14 pH relationship between the influent, effluent and percentage change in efficiency For the treatment plant from January-December, 2014

An annual relationship between the pH for the treatment plant is as shown in Fig. 4.14 above. The percentage change in efficiency for the year 2014 as shown in the chart above depict that the change is minimal ranging for 0 to 4%, indicating that the system is stable in terms of its pH value.

The pH of the wastewater during the study remained between 6.99 and 7.26. Extremes of pH are fatal for most bacteria. The bacteria grow best when the pH is slightly on the acidic side. The optimum range for bacterial growth generally lies between 6.5 and 7.5 (Devendra and Mahesh, 2014).

4.1.13 Electrical conductivity (EC) removal

Table 4.15 Average Electrical Conductivity (EC) characteristics of the treatment plant from May-Sept., 2015

Period		Influent (μS/cm)	Effluent (μS/cm)	Removal efficiency (%)	On river WUPA	
					10m upstream (μS/cm)	10m downstream (μS/cm)
May-Sept., 2015	Mean	273.65	208.90	10.50	215.17	222.25
	S.D	43.45	21.43	10.08	17.58	23.22
	SKEW	-0.45	0.13	1.23	2.16	0.77
W.H.O standards		1250				
NESREA Effluent limits		600				

The electrical conductivity of the wastewater during the research period is as contained in the table above (Table 4.15); the influent had an average value of 273.65μS/cm, standard deviation of which is 43.5 signifying a wide range of variation in the values having skewness coefficient of -0.45.

The effluent recorded an average value of 208.90μS/cm which has standard deviation of 21.43 and skewness coefficient of 0.13, resulting in removal efficiency of 11%.

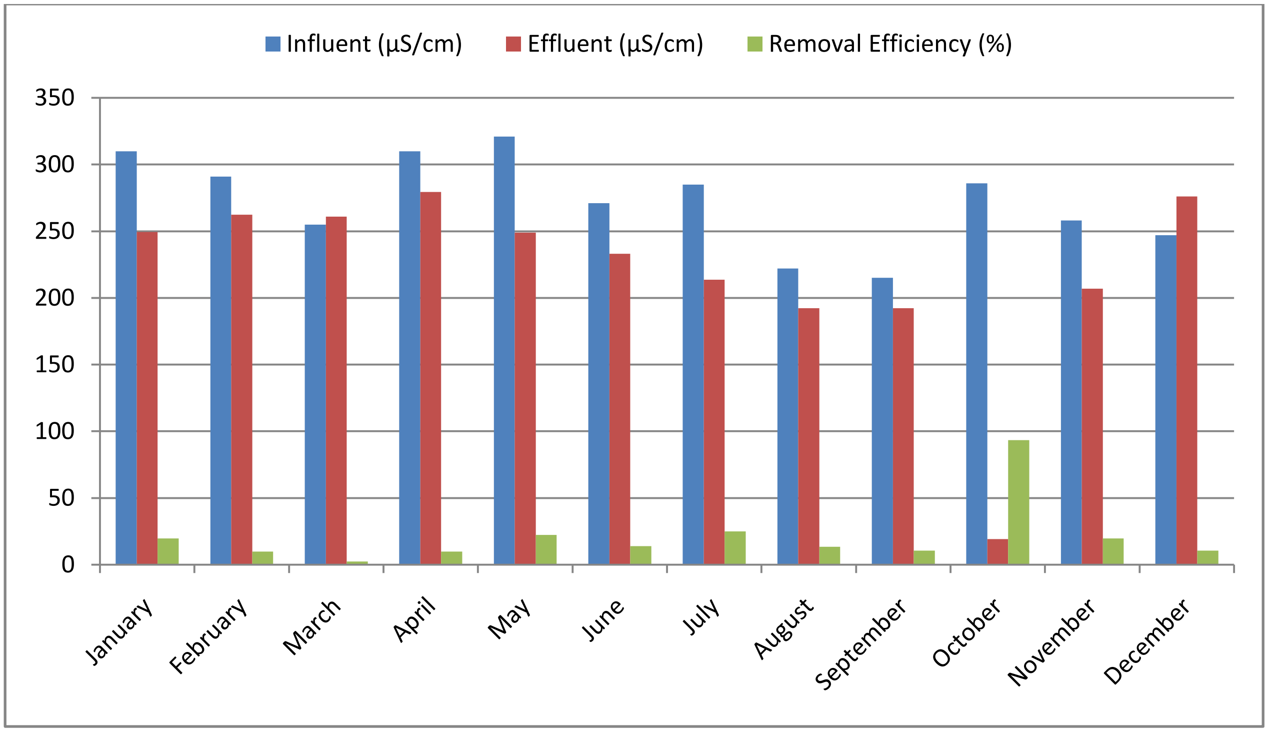


Fig. 4.15 EC relationship between the influent, effluent and removal efficiency for the treatment plant from January-December, 2014

The figure above (Fig. 4.15) is the graphical representation of the relationship between the influent, effluent and removal efficiency in terms of electrical conductivity for the period from January to December, 2014. The month of October recorded 93% removal efficiency which is the highest while March recorded 2% removal efficiency which is the least throughout the year.

4.2 Impacts of effluent from WUPA sewage treatment plant on river WUPA

The results of this research in terms of physicochemical and bacteriological parameters are as shown in Tables 4.1 to Table 4.15, this section assess the environmental impact of the waste effluents on the adjoining river.

The average BOD_5 of the effluent from the treatment plant was 9.33 mg/l (Table 4.1) being lower than the average BOD_5 values for the receiving water body (river WUPA) at 10m upstream and 10m downstream of 13.50 mg/l and 12.50 mg/l respectively. Moreover these values are all within the permissible limits of 30 mg/l for both WHO and NESREA.

Likewise the COD of the effluent was 46.41 mg/l (Table 4.2); it is within the permissible limits of 100 mg/l for WHO and 80 mg/l for NESREA. The average COD values for the sampled points at 10 m upstream and 10 m downstream were 42.25 mg/l and 40.66 mg/l respectively. Though the COD values of the effluent was higher than both the upstream and downstream values, it may have not impacted the receiving water body negatively as these values are still within the discharge limits for both WHO of 100 mg/l and NESREA of 80 mg/l. The high COD value is attributable to the availability of inorganic substances in the wastewater.

Most water bodies and wastewater alike contains high concentration of Total Suspended Solids (TSS). The effluent from the treatment plant had an average TSS value of 12.58 mg/l, which is less than the specified values of 30 mg/l for both WHO and NESREA as discharge limits. The receiving water body (River WUPA) had average TSS values of 30 mg/l and 29.33 mg/l at the upstream and downstream respectively (Table 4.5). The TSS was observed to have decreased at the downstream indicating that the effluent discharged from the treatment plant had diluted the receiving water body. Furthermore, it can be said that the effluent from the treatment plant had been used to lower the TSS of the river.

The dissolved oxygen (DO) content of the effluent water was observed to averaged 7.44 mg/l which is within the specified limits of 7-10 mg/l for both WHO and NESREA (Table 4.6). The DO for the river at 10 m upstream and 10 m downstream was observed to be 6.54 and 7.24 mg/l respectively, hence the discharged effluent from the treatment plant has not impacted the receiving water body and the aquatic community in terms of DO negatively.

In terms of Nitrate (NO_3-N) and Nitrite (NO_2-N), the average effluent values were observed to be 3.40 mg/l and 0.42 mg/l respectively (Table 4.7 and Table 4.8). The Nitrate value of 3.40 mg/l which seems to have increased after treatment was within the discharge limits of 10 mg/l for WHO and 20 mg/l for NESREA with values of 1.70 and 1.47 mg/l for the upstream and downstream respectively.

The WHO discharge limits for Nitrite is 2 mg/l and 1 mg/l for NESREA, concentration of Nitrite at 10 m upstream and 10 m downstream were 0.71 mg/l and 0.53 mg/l respectively indicating that the discharged effluent from the treatment plant has not impacted the river negatively.

Ammonium as Nitrogen (NH_4-N) for the treatment plant and the receiving river was monitored and observed on the average to be 0.79 mg/l at the effluent outlet (Table 4.9), and at sampled points 10 m upstream and 10 m downstream, the concentrations were observed to be 1.35 mg/l and 0.88 mg/l respectively. For Chloride (CL^-), the average concentration at the effluent outlet was observed to be 20.18 mg/l (Table 4.10) while at 10 m upstream and 10 m downstream, the concentration was 17.20 mg/l and 17.53 mg/l respectively being lower than the discharged limits of 250 mg/l for WHO and 600 mg/l for NESREA. Furthermore, the effluent from the treatment plant does not impact the receiving water body and environment negatively in terms of Chloride.

Moreover, Sulfate (SO_4^-) effluent concentration averaged 42.08 mg/l (Table 4.1) and at sampled points 10 m upstream and 10 m downstream the values were 47.75 mg/l and 43.75 mg/l respectively which are all within the specified limits for WHO of 250 mg/l and 500 mg/l for NESREA.

Total coliform count (TCC) and faecal coliform (FC) were used as an indicator parameter for the potential removal of bacteriological contamination. The result of this research showed that wastewaters are treated to eliminate pathogenic microorganisms and to prevent waterborne transmission using ultra violet (UV) radiation.

For TCC, the effluent had an average of 128 cfu/100ml which is higher than the 94 and 82 cfu/100ml observed for sampled points at the upstream and downstream respectively (Table 4.12) while for FC, the effluent after UV had an average value of 38 MPN/100ml with upstream and downstream samples having 26 and 25 MPN/100ml respectively (Table 4.13).

Therefore, wastewater treatment reduces but does not guarantee the complete elimination of a putative contamination with bacteria. This was in accordance with the finding of Salem *et al*. (2011). Downstream samples had lower percentage of contamination because of self recovery activity of the stream, although this does not guarantee complete absence of bacterial contamination.

The average pH values recorded for the effluent and the sampling points were 7.04 for the effluent, the upstream and downstream values are 7.22 and 7.16 respectively which are within the WHO pH tolerance limits of between 6.5-8.5 though slightly above the NESREA limits of 6.9 (Table 4.14).

The electrical conductivity limits specified by WHO is 1250 µs/cm while that of NESREA is 600 µs/cm, average value obtained from this study for the effluent is 208.90 µs/cm and that of the sampled points at upstream and downstream of the receiving water body are 215.17 and 222.25 µs/cm respectively (Table 4.15), hence the observed values are within the specified limits.

4.3 Maintenance plan for WUPA sewage treatment plant

The WUPA sewage treatment plant Abuja (WSTP) is an activated sludge process, treating domestic wastewater generated from the city center (Abuja). Due to the importance accorded the treatment plant and considering the erratic power supply of the nation, the treatment plant is powered by two number (1500KVA and 1350KVA) generating plants which are used alternatively.

The efficiency of the treatment plant was not 100%, therefore the need to develop a maintenance plan. Most importantly, the observed presence of total coliform count and faecal coliform at the effluent outlet obviates the need to properly maintain the UV light for maximum performance.

Table 4.16 shows the inspection required for screens and grits removal, primary sedimentation tank, aeration tank and secondary sedimentation tank units operations of the treatment plant and the frequency of maintenance needed for effective operation. Clause 4.3.1 to 4.3.7 gives a brief description of components of the Trojan UV3000™ used in the treatment plant with a view to properly understand its scheduled maintenance as in Table 4.17 and Table 4.18 summarized the inspections required specifying the frequency of the operations for effective maintenance of the dewatering area.

Table 4.16 Maintenance plan for units processes at Wapa Sewage Treatment Plant, Abuja

	Process /units	Inspection required	Frequency
1	Screens and Grits Removal	Need for visual inspection, cleaning and greasing	Daily
		Need for effective alarms for blockages, high levels, machines failure	Daily
		Any problems, including shortfall in	Daily

		capacity, in the screens and grits removal will cause downstream problems	
2	Primary Sedimentation Tank	Sweeping of channels and scrapper arms	Daily
		Ensure scrapers and scum removal are in good order	Daily checks
		Periodic drain down and cleaning	Daily checks
3	Aeration tank	Typically 50-70% of site energy use	Monthly
		Install sub-metering	Weekly
		Carry out overall efficiency measurements. Target values: • 1.5 kilograms O_2/kilowatt hour (kWh) for surface aeration • 2.5 kg O_2 kWh for fine bubble diffused aeration	Weekly
		Use variable speed drives (VSD) rather than throttling flows to control aeration	Weekly
		Minimize necessary biomass to treat full load to avoid aerating excess sludge	Weekly
		For simple feedback DO control: • Minimize DO set points • Understand DO profile in tank, avoid over aeration	Daily
		Most plants use fixed dissolved oxygen (DO) set points, typically 1.5-2 milligrams/litre (mg/l)	Daily
		Check on DO profile across tank	Daily
4	Secondary Sedimentation tank	DO probes still need careful cleaning	Weekly
		Calibration check	Monthly
		Cleaning of diffusers/membranes and replace when pressure loss excessive	Monthly
		Check pressure drop across air filter, replace when pressure loss is excessive	Daily
		Inspect tank surface patterns for signs of blocked/damage diffusers	Daily
		Main blowers	Weekly
		Cleaning of distribution arms and nozzles	Daily

5	Works flow meters	Electromagnetic meters: • Cleaning • Calibration	At least annual, 2-3 years interval
		Open channel meters: • Cleaning of flow structure (flume) channel; • Calibration of level meters	Weekly Monthly
6	Odour Control	Channels and tanks often covered	Always
		Maintain gas scrubbing system	Always
		Check achieved flow rates	Daily
		Check hydrogen sulphide gases	Daily
7	Biomass Management	Good treatment need a healthy biomass	Weekly check
		Need to measure: • Mixed liquor suspended solid (MLSS) • Returned activated sludge (RAS) solids • Stirred Specific Volume Index (SSVI) • RAS how (Returned activated sludge) • Microscopy, Respirometry.	Daily check
8	Good practice	Make path as short as possible	
		Temperature sensors needs to be shaded from direct sunlight, fit shade if necessary	

4.3.1 Ultraviolet disinfection

Microorganisms in the treated wastewater are exposed to ultra violet light when they pass through special lamps. The UV energy instantly destroys the genetic material (DNA) within bacteria, viruses and protozoa, eliminating their ability to reproduce and cause infection, unable to multiply, the microorganisms die and no longer pose a health risk.

The Trojan UV3000™ is made up of several components:

• UV Module

• Electronic ballast

• UV Sensor

• Power distribution center (PDC)

• System control center (SCC)

• Automatic level control (ALC)

4.3.2 UV Module

The UV module is the basic unit of the flow through UV bank. A bank is made up of 24 UV modules placed in parallel, 3 inches apart. UV modules consist of a 316 stainless steel frame that holds 8 high-intensity UV lamps in position, and houses all connecting wires and electronic ballasts in a watertight enclosure.

4.3.3 Electronic ballast

The ballast is mounted within a watertight enclosure on top of the module frame. There is no need for mechanical cooling since normal convection cooling is adequate.

4.3.4 UV Sensor

The submersible UV Sensor measures the UV intensity within each bank of UV lamp modules. The UV Sensor is mounted on a representative UV lamp module. The UV Sensor is calibrated in the factory and should not be altered, or its calibration changed.

4.3.5 Power distribution center (PDC)

The Power Distribution Center spans the width of the effluent channel and distributes power from the main electrical service to the UV modules in the UV banks. Molded connectors connect the UV modules to the PDC by plugging them into the stainless steel receptacles on the PDC's front panel. The PDC is a stainless steel enclosure that is weather resistant. It houses the power distribution bus bar, relay board for each module, and the communication controller board. The main power service is connected through the electric service entrance power and distributed through the bus bar. Communications between the PDC and System Control Center is via an RS422 Serial Communication Link.

4.3.6 System control center (SCC)

The operation of the Trojan UV3000™ is managed by the system control center (SCC). The SCC is a menu-driven workstation that supplies the operator interface to the disinfection system. It allows the operator to monitor and control all UV system functions. An alarm reporting system provides the operator with the tools needed for accurate diagnosis of various system processes and failures.

4.3.7 Water level control

An Automatic level controller (ALC) device controls the effluent level within the UV channel.

Table 4.17 Equipment maintenance schedule-UV

PROCESS/UNIT	Inspection required	Frequency
Ultra Violet Light (UV)	Check bank status, alarm status and UV intensity status screens for any new faults. Record	Daily
	Check for debris build-up on modules leg or in the channel	Daily
	Check and record lamp hours	Weekly
	Check electronic ballast, replace if necessary	Monthly
	Clean any algae or debris build-up from sensor	Monthly
	Clean quartz sleeve from algae build-up, housing off the sleeves may be all that is required, but a coating will build-up over time in which case a thorough cleaning will be necessary	Monthly
	Clean the system control center (SCC) enclosure. Do not use high- pressure hose or corrosive cleaners	Monthly
	Check system control center (SCC) seal. Ensure moisture is not present	Monthly
	Clean the power distribution center (PDC) enclosure. Do not use high-pressure hose or	Monthly

	corrosive cleaners	
	Check module cables. Ensure module cables are tightly mated to female receptacles	Monthly
	Check level control device for algae build-up hose off if necessary	Monthly

Table 4.18 Equipment inspection schedule-dewatering area

Equipment/ Item	**Inspection required**	**Frequency**
	Gravity thickeners	
Gravity Thickener Tank	Flanges	Daily
	Valves	Daily
	Piping	Daily
	Solids content	Daily
	Liquid levels	Daily
Rake Arm/Drive Unit	Drive motor	Daily
	Couplings	Daily
	Gear box	Daily
	Flanges	Daily
	Drive chains/sprockets	Daily
Rake Arm Lift Mechanism	Worm gear	Monthly
	Outer shaft sprocket	Monthly
	Drive motor	Monthly
	Bearings	Monthly
	Performance	Monthly
Thickened Underflow Pumps	Motor	Daily
	Guards	Daily
	Pump	Daily
	Seals	Daily
	Bearings	Daily
	Lubrication	Daily
	couplings	Daily
	Discharge pressure	Daily
Thickened Slurry Tanks	Piping	Daily
	Valving	Daily
	Levels	Daily
	Inspection gates	Daily
	Flanges	Daily
	Ladders	Daily
	Handrails	Daily
Thickened Slurry Tank Mixers	Motor	Daily
	Gear box	Daily
	Bearings	Daily
	Seals	Daily

	Couplings	Daily
	Shaft flanges	Daily
	Lubrication	Daily
Filter presses		
Filter Press Feed Pumps	Motors	Daily
	Pumps	Daily
	Bearings	Daily
	Couplings	Daily
	Seals	Daily
	Discharge pressure	Daily
	Lubrication	daily
Dewatering Building Compressed Air System Compressor	Motor	Daily
	Compressor fluids	Daily
	Belts	Daily
	Sheaves	Daily
	Pulleys	Daily
	Pressure relief valves	Daily
	Operating pressures	Daily
	Air intake filters	Daily
	Condensation drains/water traps	Daily
	Air dryers	Daily
Filter Press Solenoid Valves	Ram pressure	Daily
	Ram seals	Daily
	Hydraulic fittings	Daily
	Motor	Daily
	Fluid levels	Daily
	Hydraulic oil filter	Daily
	Operating performance	Daily
Polymer feed system		
Neat Polymer Storage Tanks	Fill line	Daily
	Vent line	Daily
	Level indicator	Daily
	Drain line	Daily
	Flanges	Daily
	Valves	Daily
Neat Polymer (Flocculant) Tank Mixer	Tank mixer motor	Daily
	Gear box	Daily
	Bearings	Daily
	Seals	Daily
	Flanges	Daily
	Couplers	Daily
	Guards	Daily
	Gear lube	Daily
Polymer Makeup Units	Metered feed system	Daily
	Metered feed pump system	Daily

	Recirculation pumps	Daily
	Piping	Daily
	Valves	Daily
	Flanges	Daily
	Couplings	Daily
Polymer Day Tank Mixer	Motors	Daily
	Guards	Daily
	Shafts	Daily
	Coupler	Daily
	Propellers	Daily
	Mounting brackets	Daily
Polymer (Coagulant) Feed Pumps	Motors	Daily
	Pumps	Daily
	Pipings	Daily
	Valves	Daily
	Bearings	Daily
	Guards	Daily
	Performance	Daily
Polymer (Flocculant) Feed Pumps	Motors	Daily
	Pumps	Daily
	Piping	Daily
	Valves	Daily
	Bearings	Daily
	Guards	Daily
	Performance	Daily
	Couplings	Daily
	Fittings	Daily
Recycle water		
Recycle Water Collection Wet Well	High level	Weekly
	Low level	Weekly
Recycle Water Collection Lift Station Pumps	Motor	Daily
	Pumps	Daily
	Bearings	Daily
	Lube	Daily
	Guards	Daily
	Piping	Daily
	Valves	Daily
	Connections	Daily
	Performance	Daily
Recycle Water Equalization Tank Feed Pumps	Motor	Daily
	Pumps	Daily
	Bearings	Daily
	Lube	Daily
	Guards	Daily
	Piping	Daily

	Valves	Daily
	Connections	Daily
	Performance	Daily
Size Separation Process Water Storage Tank Feed Pumps	Motors	Daily
	Pumps	Daily
	Bearings	Daily
	Lube	Daily
	Guards	Daily
	Piping	Daily
	Valves	Daily
	Connections	Daily
	Performance	Daily
Filter cake solids enclosure		
Solids Enclosure (Membrane Panel; Frame)	Anchor bolt	Annual
	Nuts	Annual
	Fasteners	Annual
	Panels	Annual
	Support beam	Annual
	Support rafters	Annual
	Bracing	Annual
	Doors	Annual
	Fabric	Bimonthly
Makeup Air Handling Units	Motors	Monthly
	Belts	Monthly
	Blower	Monthly
	Bearings	Monthly
	Sheaves	Monthly
	Pulleys	Monthly
	Guards	Monthly
	Housing	Monthly
	Duct work	Monthly
	Vents/outlets	Monthly
	Flow control dampers	Monthly
Filter cake solids enclosure		
Exhaust Fans	Motors	Annual
	Blowers	Annual
	Shafts	Annual
	Bearings	Annual
	Guards	Annual
	Duct	Annual
	Dampers	Annual
Filters	Filter media	monthly

Source: Parson 2009

4.4 Modeling activated sludge process

Modeling the activated sludge process has an important role in implementing efficient control actions for better process performance. Models are helpful mainly because the effects of adjusting the operating variables can be studied far more quickly on a computer than by doing experiments as illustrated in this study. Hence, many alternative designs and operational strategies can be compared without the need for physical trials of each scenario (Rabee, 2009). By simulating this model with the possible correction actions, it is then possible to rapidly respond to any change in the process and devise an operational strategy, which can move the plant to new operating conditions that improve its stability, the quality of the effluent and at the same time achieve reduction in the running costs. Therefore, optimum process configurations, which meet given effluent quality standards at least cost, can be achieved (Olson *et al.*, 2005; Rivas et al., 2008).

4.4.1 Simple Mathematical Model for WUPA Sewage Treatment Plant Abuja

The average values for Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS) for the treatment plant were plotted as shown in Fig. 4.5 and Fig. 4.6 which indicated that the equation for the curve is quadratic in nature; hence $Y = Ax^2 + Bx + C$

Where A, B and C are constants, Y is the dependent variable representing the BOD and TSS at time t and x represent the hydraulic retention time (HRT) in hours for the treatment as applicable.

Table 4.20 is the average result of BOD_5 and TSS with their corresponding hydraulic retention time (HRT).

Table 4.19 Results of average BOD_5, TSS and hydraulic retention time for the treatment plant

S/NO	Unit Processes	Av. BOD_5 (mg/l)	Av. TSS (mg/l)	Av. HRT (hr.)	Cum. HRT (hr.)
1	Inlet to Treatment Plant (SCP 1)	126.08	146.91	0.00	0.00
2	Outlet of aeration basin (SCP 7C-2)	37.33	289.25	41.40	41.40
3	Outlet of sedimentation (SCP 8C-3)	9.50	15.16	7.00	48.40
4	Effluent outlet after UV (SCP 4)	9.33	12.58	1.00	49.40

4.4.2 Relationship between average BOD_5 and sample collection points along the treatment plant

The BOD of the wastewater decreases as the treatment progresses along the treatment plant

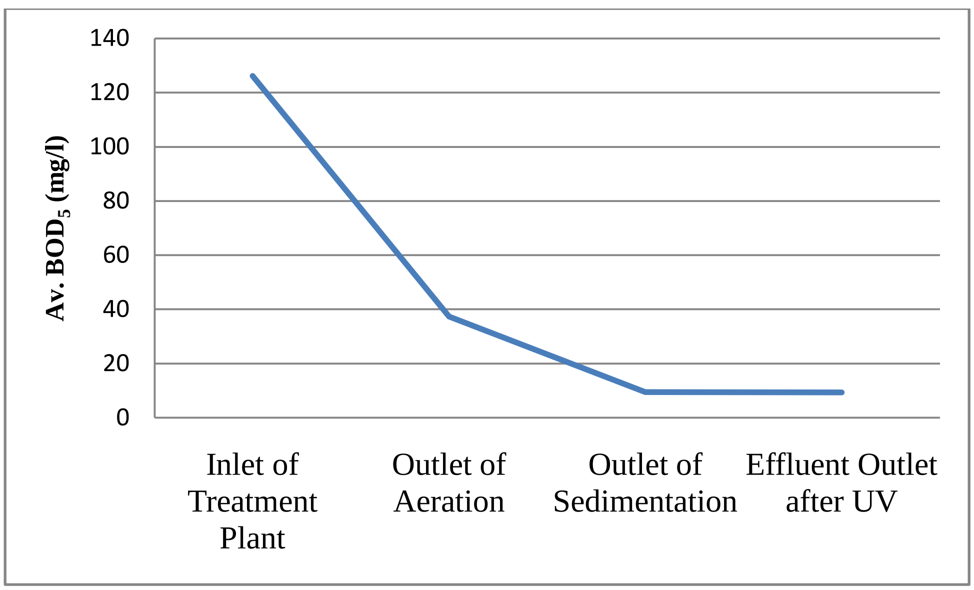


Figure 4.16 Average BOD_5 against sample collection points along the treatment plant

4.4.3 Relationship between average TSS and sample collection points along the treatment plant

The TSS of the wastewater tends to increase at the outlet of the aeration and decrease along the treatment process as shown below.

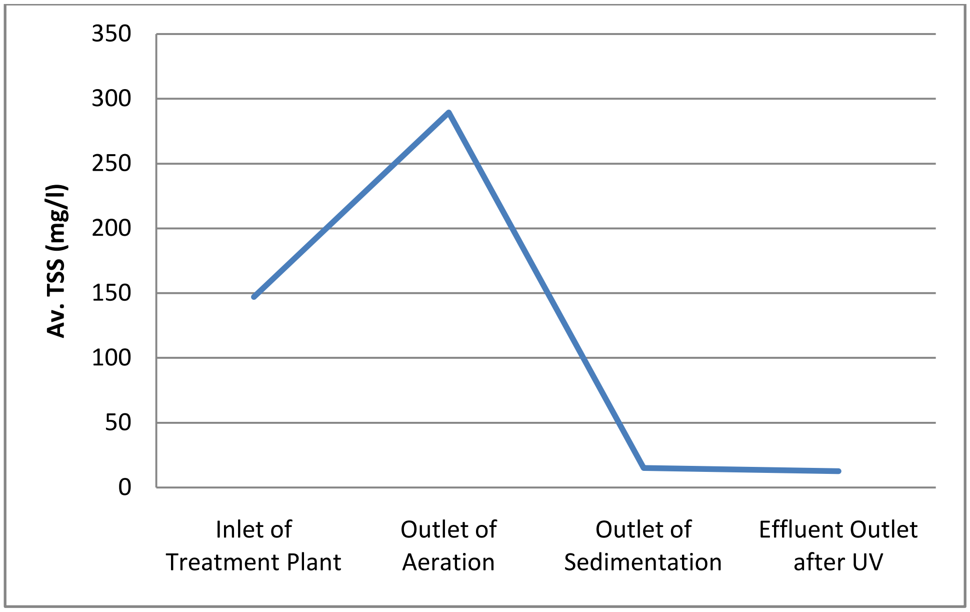


Figure 4.17 Average TSS against sample collection points along the treatment plant.

By calculation, a model for the BOD_5 and TSS is developed as shown below using the quadratic equation $Y = Ax^2 + Bx + C$ 5.1

For BOD_5

When x = 0hrs.

$126.08 = A (0)^2 + B (0) + C;$ $C = 126.08$

When x = 41.4hrs.

$37.33 = A (41.4)^2 + B (41.4) + 126.08$

$1713.96A + 41.40B = -88.75$ 5.2

When x = 7hrs.

$9.5 = A (7)^2 + B (7) + 126.08$

$49A + 7B = -116.58$ 5.3

Solving for A and B from equation 5.2 and 5.3

$A = 0.4218; B = -19.6070$

From the results obtained, a model of BOD as a function of hydraulic retention time x was presented below

$Y = 0.4218x^2 – 19.6070x + 126.08$ 5.4

For TSS

When x = 0hrs.

$146.91 = A (0)^2 + B (0) + C;$ $C = 146.91$

When x = 41.4hrs.

$289.25 = A (41.4)^2 + B (41.4) + 146.91$

$1713.96A + 41.40B = 142.34$ 5.5

When x = 7hrs.

$15.16 = A (7)^2 + B (7) + 146.91$

$$49A + 7B = -131.75 \tag{5.6}$$

Solving for A and B from equation 5.5 and 5.6

$A = 0.6471$; $B = -23.3510$

Also from the results obtained, a model of TSS as a function of hydraulic retention time x was presented below

$$Y = 0.6471x^2 - 23.3510x + 146.91 \tag{5.7}$$

CHAPTER FIVE

SUMMARY, CONCLUSION AND RECOMMENDATIONS

5.1 Summary

This study was conducted in WUPA sewage treatment plant (WSTP) in Abuja. The treatment plant was established to treat domestic wastewater generated from the capital city without consideration for treating industrial wastewater. As the WSTP is a biological process, it has its own capacity to treat wastewater. Therefore this study aimed to evaluate the performance of the plant in terms of five day biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), total suspended solids (TSS), nitrate as nitrogen (NO_3-N), nitrite as nitrogen (NO_2-N), ammonia as nitrogen (NH_4-N), dissolved oxygen (DO), chloride (CL^-), sulfate (SO_4^-), faecal coliform (FC), total coliform count (TCC), pH and electrical conductivity (EC) removal/increase.

Furthermore, the performance of the WSTP depends on the characteristics of influent into the treatment plant. Considering that there are settlements at the downstream of the treatment plants that used the river for either domestic or fishing purposes, hence the need to determine the impact of the treated effluent from the plant on the river and since the treatment plant incorporates both mechanical and biological process, there is the need for its proper maintenance.

In chapter four, the results of finding from this study are presented, indicating the percent of removal/increase for each of the aforementioned parameters. Maximum removal efficiency of 92% was achieved for BOD_5 while COD and TSS recorded removal efficiency of 83 and 89% respectively. For CL^-, SO_4^-, pH and EC percent removal recorded are 19, 30, 3 and

11% respectively. Some of the parameters rather recorded increase in their concentrations; the parameters and the correspondence percent increase are 46% for DO, 60% for NO_3-N and 72% for NO_2-N.

Throughout the twelve weeks period of sampling and analysis for this study, it was observed that the treated effluent from the treatment plant has no negative impact on the receiving water body except for total coliform count and faecal coliform which had percent removal of greater than 91% and 93% respectively. Furthermore, it was observed that the removal efficiency of the treatment plant in terms of the parameters tested was not 100%, therefore the need to evolve a maintenance plan for the sewage treatment plant.

5.2 Conclusion

This research work has revealed that at the time of conducting this work, the treatment plant was operating at 30% designed capacity as a result of the low volume of wastewater discharged into the treatment plant and that activated sludge process is very effective in treating domestic wastewater as observed in the high removal efficiency for biochemical oxygen demand (BOD), chemical oxygen demand (COD), total suspended solids (TSS), total coliform count (TCC) and faecal coliform (FC).

The models for BOD and TSS for the WUPA sewage treatment plant are:

$Y = 0.4218x^2 – 19.6070x + 126.08$ and

$Y = 0.6471x^2 – 23.3510x + 146.91$ respectively.

Though the World Health Organization (W.H.O) and National Environmental Standards Regulation and Enforcement Agency (NESREA) discharge limits does not specify the acceptable limits for total coliform count (TCC) and faecal coliform (FC), it was discovered during this study that the bacteriological load for the effluent in terms of TCC and FC were higher than the values for both downstream and upstream of the receiving river.

It was also discovered during the course of this research work that the treatment plant is being operated by power generating plants which the researcher conclude that the cost incurred in running the plant is exorbitant in relation to been powered by the Power Holding Company of Nigeria (PHCN).

5.3 Recommendations

The bacteriological (total coliform count and faecal coliform) qualities of the treated water (effluent) was observed to be higher than that of the receiving water body (river WUPA) thereby having tendency of causing waterborne diseases to human beings that consume the water downstream of the treatment plant. There is the need to ensure that the ultraviolet light (UV) is functioning effectively to reduce the microbial load of the effluent water.

There is the need for the intervention of appropriate authorities to monitor the kinds of waste discharged into the river as the concentration of the water sampled on the river in terms of the physicochemical parameters monitored was high compared to that of the effluent water.

To overcome the problem of under loading and utilization of the treatment plant in terms of its designed capacity (BOD_5 and TSS), other factory sewerages systems should be pretreated and connected to the WSTP.

Further research is recommended to determine the presence of heavy metals in the effluent water and a complete model incorporating all parameters should be developed for the treatment plant. The possibility of generating hydroelectricity from the effluent discharged into the river should be looked into instead of operating the plant solely on power generating plants. Moreover a research should be conducted into the generation of biogas from the sludge generated from the treatment plant.

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APPENDIX A

Table A.1 BOD$_5$ (mg/l) characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Removal efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	160.00	60.00	16.00	16.00	90.00	25.00	20.00
Week 2	170.00	70.00	13.00	12.00	92.94	15.00	16.00
Week 3	180.00	80.00	10.00	10.00	94.44	15.00	12.00
Week 4	180.00	60.00	8.00	8.00	99.56	18.00	20.00
Week 5	200.00	50.00	10.00	10.00	95.00	19.00	17.00
Week 6	160.00	60.00	8.00	8.00	99.50	15.00	16.00
Week 7	75.00	12.00	8.00	8.00	89.33	12.00	10.00
Week 8	80.00	13.00	10.00	10.00	87.50	8.00	9.00
Week 9	78.00	10.00	6.00	6.00	92.30	8.00	8.00
Week 10	85.00	12.00	8.00	8.00	90.58	9.00	6.00
Week 11	70.00	10.00	8.00	8.00	88.57	9.00	8.00
Week 12	75.00	11.00	9.00	8.00	89.33	9.00	8.00
Mean	126.08	37.33	9.50	9.33	92.42	13.50	12.50
S.D	52.22	28.05	2.68	2.61	4.04	5.33	5.04
SKEW	0.12	0.21	0.21	1.46	1.63	0.81	0.85
W.H.O standards				30			
NESREA Effluent limits				30			

Table A.2 COD (mg/l) characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Removal efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	334.00	120.00	60.00	60.00	82.04	50.00	55.00
Week 2	369.00	129.00	63.00	63.00	82.93	60.00	59.00
Week 3	400.00	130.00	60.00	60.00	85.00	55.00	56.00
Week 4	380.00	120.00	50.00	55.00	85.53	50.00	50.00
Week 5	395.00	100.00	55.00	50.00	87.34	55.00	50.00
Week 6	350.00	80.00	60.00	55.00	84.29	50.00	50.00
Week 7	210.00	95.00	45.00	45.00	78.57	35.00	30.00
Week 8	200.00	80.00	42.00	42.00	79.00	32.00	30.00
Week 9	195.00	69.00	35.00	35.00	82.05	30.00	28.00
Week 10	200.00	60.00	30.00	30.00	85.00	28.00	25.00
Week 11	190.00	60.00	32.00	32.00	83.15	30.00	25.00
Week 12	180.00	55.00	30.00	30.00	83.33	32.00	30.00
Mean	283.58	91.50	46.83	46.41	83.18	42.25	40.66
S.D	93.53	28.10	12.83	12.44	2.56	12.01	13.59
SKEW	0.09	0.15	-0.15	-0.14	-0.50	0.13	0.08
W.H.O standards				100			
NESREA Effluent limits				80			

Table A.5 TSS (mg/l) characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Removal efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	160.00	355.00	29.00	23.00	85.63	44.00	34.00
Week 2	180.00	400.00	20.00	13.00	92.78	43.00	39.00
Week 3	165.00	460.00	25.00	16.00	90.30	50.00	30.00
Week 4	170.00	470.00	16.00	15.00	91.18	60.00	50.00
Week 5	230.00	290.00	13.00	12.00	74.78	50.00	50.00
Week 6	200.00	290.00	12.00	12.00	94.00	50.00	40.00
Week 7	115.00	215.00	15.00	12.00	89.56	25.00	19.00
Week 8	108.00	215.00	12.00	10.00	90.74	28.00	22.00
Week 9	120.00	230.00	12.00	10.00	91.66	30.00	20.00
Week 10	110.00	208.00	10.00	8.00	92.72	24.00	15.00
Week 11	105.00	170.00	10.00	12.00	88.57	20.00	18.00
Week 12	100.00	168.00	8.00	8.00	92.00	20.00	15.00
Mean	146.91	289.25	15.16	12.58	89.49	37.00	29.33
S.D	43.09	108.07	6.41	4.08	5.13	13.95	13.04
SKEW	0.60	0.65	1.23	1.54	-2.46	0.18	0.50
W.H.O standards				30			
NESREA Effluent limits				30			

Table A.6 DO (mg/l) characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Removal efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	1.90	4.30	5.90	6.80	72.06	5.50	6.50
Week 2	2.50	4.50	6.00	7.00	64.29	5.60	6.20
Week 3	2.90	4.00	6.00	6.90	57.97	6.00	6.40
Week 4	3.00	3.50	7.00	7.00	57.14	6.00	6.20
Week 5	4.00	3.50	6.50	6.50	38.46	6.20	6.40
Week 6	2.50	3.00	6.60	6.40	60.94	6.00	6.20
Week 7	5.00	5.50	7.00	8.00	37.50	7.00	8.00
Week 8	4.90	5.50	6.00	7.90	37.97	7.50	8.50
Week 9	5.50	6.00	7.60	8.00	31.25	6.80	7.50
Week 10	6.00	7.00	7.50	8.50	29.41	6.90	8.00
Week 11	5.50	6.00	7.50	7.80	29.48	7.50	8.50
Week 12	5.00	6.50	7.00	8.50	41.17	7.50	8.50
Mean	4.05	4.94	6.71	7.44	46.47	6.54	7.24
S.D	1.43	1.31	0.64	0.75	15.03	0.75	1.01
SKEW	-0.18	0.04	0.02	0.08	0.41	0.08	0.20
W.H.O standards				7-10			
NESREA Effluent limits				7-10			

Table A.7 NO_3-N (mg/l) characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Increase in efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	1.00	1.00	1.20	1.20	16.67	1.00	1.10
Week 2	1.50	8.50	4.50	4.00	62.50	1.20	1.20
Week 3	1.20	7.50	3.90	3.00	60.00	1.50	1.20
Week 4	1.20	5.50	3.60	2.50	52.00	1.30	1.20
Week 5	0.90	4.50	4.00	3.50	74.29	1.90	1.20
Week 6	0.70	4.50	4.00	3.90	82.05	2.00	1.30
Week 7	1.50	7.50	6.00	2.50	40.00	1.50	1.70
Week 8	1.45	8.00	3.50	4.00	63.75	2.00	2.20
Week 9	1.20	7.00	2.00	3.50	65.71	1.00	1.00
Week 10	1.25	5.00	2.50	3.10	59.67	1.80	1.50
Week 11	1.35	6.50	3.50	4.20	67.85	2.80	2.10
Week 12	1.00	6.00	2.30	5.50	81.81	2.50	2.00
Mean	1.18	5.95	3.41	3.40	60.52	1.70	1.47
S.D	0.25	2.06	1.27	1.07	18.14	0.57	0.42
SKEW	-0.48	-1.19	0.17	-0.19	-1.29	0.55	0.78
W.H.O standards				10			
NESREA Effluent limits				20			

Table A.8 NO_2-N (mg/l) characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Increase in efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	0.25	3.50	1.50	1.00	75.00	1.00	1.00
Week 2	0.29	2.50	0.90	0.80	63.75	0.70	0.70
Week 3	0.19	2.60	0.70	0.50	62.00	0.60	0.50
Week 4	0.12	2.70	0.60	0.60	80.00	0.30	0.20
Week 5	0.23	2.90	0.80	0.70	67.14	0.70	0.50
Week 6	0.39	2.80	0.70	0.80	51.25	0.60	0.40
Week 7	0.50	3.00	0.50	0.10	80.00	1.20	1.00
Week 8	1.00	3.50	0.80	0.06	94.00	0.80	0.20
Week 9	0.45	3.20	0.80	0.10	77.77	0.70	0.50
Week 10	0.70	2.80	0.50	0.15	78.57	0.80	0.50
Week 11	0.45	3.20	0.40	0.15	66.66	0.60	0.40
Week 12	0.40	2.80	0.50	0.15	62.50	0.60	0.50
Mean	0.41	2.95	0.72	0.42	71.55	0.71	0.53
S.D	0.24	0.33	0.29	0.34	11.43	0.22	0.26
SKEW	1.34	0.54	1.80	0.38	0.18	0.56	0.83
W.H.O standards				2			
NESREA Effluent limits				1			

Table A.9 NH_4-N (mg/l) characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Removal efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	12.00	0.50	0.35	0.35	97.08	1.50	0.95
Week 2	6.50	1.50	0.49	0.40	93.85	1.30	0.85
Week 3	4.90	2.00	0.24	0.21	95.71	1.60	0.99
Week 4	4.90	2.00	0.25	0.22	95.51	1.70	0.98
Week 5	5.00	4.00	0.36	0.30	94.00	1.60	0.85
Week 6	6.00	5.00	0.29	0.24	96.00	1.70	0.80
Week 7	2.00	1.40	1.20	1.50	25.00	0.90	0.90
Week 8	2.00	1.50	1.20	1.30	35.00	1.00	0.80
Week 9	1.80	1.30	1.30	1.20	33.33	1.20	0.85
Week 10	1.50	1.20	1.00	1.30	13.33	1.00	0.95
Week 11	1.50	1.30	1.10	1.30	13.33	1.20	0.90
Week 12	1.35	1.50	1.00	1.25	7.40	1.50	0.80
Mean	4.12	1.93	0.73	0.79	58.29	1.35	0.88
S.D	3.15	1.28	0.43	0.54	39.49	0.29	0.07
SKEW	1.47	1.75	0.05	0.03	-0.13	-0.28	0.19
W.H.O standards				-			
NESREA Effluent limits				-			

Table A.10 CL^- (mg/l) characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Removal efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	22.20	23.60	22.50	23.70	6.34	20.90	20.80
Week 2	27.50	22.90	22.60	22.00	20.00	20.50	20.10
Week 3	35.40	22.50	27.70	23.00	35.03	19.00	20.00
Week 4	37.90	35.50	25.50	27.50	27.44	20.50	20.50
Week 5	27.70	25.70	22.60	23.50	15.16	17.50	17.50
Week 6	25.90	30.00	28.90	25.50	1.54	17.00	16.50
Week 7	20.00	18.00	16.00	16.00	20.00	15.00	16.00
Week 8	20.00	18.00	15.00	15.00	25.00	16.00	15.00
Week 9	19.00	17.00	15.00	16.00	15.78	15.00	17.00
Week 10	21.00	18.00	16.00	16.00	23.80	14.00	15.00
Week 11	20.00	19.00	15.00	16.00	20.00	15.00	16.00
Week 12	22.00	20.00	19.00	18.00	18.18	16.00	16.00
Mean	24.88	22.51	20.48	20.18	19.02	17.20	17.53
S.D	6.27	5.61	5.18	4.45	8.95	2.46	2.20
SKEW	1.22	1.32	0.35	0.26	-0.37	0.44	0.49
W.H.O standards		250					
NESREA Effluent limits		600					

Table A.11 SO_4^- (mg/l) characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Removal efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	75.00	60.00	60.00	55.00	26.67	60.00	55.00
Week 2	70.00	60.00	59.00	55.00	21.43	65.00	55.00
Week 3	75.00	55.00	49.00	50.00	33.33	55.00	54.00
Week 4	55.00	50.00	50.00	50.00	9.09	60.00	55.00
Week 5	55.00	50.00	50.00	45.00	18.18	50.00	45.00
Week 6	75.00	70.00	60.00	55.00	26.67	55.00	50.00
Week 7	50.00	45.00	38.00	28.00	44.00	40.00	38.00
Week 8	52.00	48.00	35.00	30.00	42.30	38.00	36.00
Week 9	55.00	48.00	40.00	35.00	36.36	35.00	34.00
Week 10	50.00	42.00	38.00	32.00	36.00	32.00	30.00
Week 11	52.00	40.00	35.00	40.00	23.07	41.00	35.00
Week 12	55.00	48.00	32.00	30.00	45.45	42.00	38.00
Mean	59.91	51.33	45.50	42.08	30.21	47.75	43.75
S.D	10.46	8.55	10.45	10.79	11.26	11.09	9.59
SKEW	0.75	0.91	0.29	-0.02	-0.30	0.14	0.07
W.H.O standards				250			
NESREA Effluent limits				500			

Table A.12 TCC (cfu/100ml) characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Removal efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	1600.00	-	1000.00	270.00	83.00	131.00	120.00
Week 2	1600.00	-	1600.00	200.00	88.00	155.00	135.00
Week 3	1600.00	-	920.00	120.00	93.00	100.00	100.00
Week 4	1600.00	-	920.00	70.00	96.00	150.00	150.00
Week 5	1600.00	-	1600.00	120.00	93.00	70.00	65.00
Week 6	1600.00	-	250.00	70.00	96.00	80.00	78.00
Week 7	1600.00	-	550.00	180.00	89.00	90.00	70.00
Week 8	1600.00	-	600.00	172.00	89.00	85.00	25.00
Week 9	1600.00	-	700.00	80.00	95.00	82.00	65.00
Week 10	1600.00	-	650.00	96.00	94.00	60.00	62.00
Week 11	1600.00	-	305.00	80.00	95.00	60.00	66.00
Week 12	1600.00	-	520.00	82.00	95.00	65.00	51.00
Mean	1600.00	-	801.00	128.00	91.00	94.00	82.00
S.D	0.00	-	437.00	63.00	4.00	34.00	37.00
SKEW	-	-	0.90	1.13	-1.19	0.95	0.61
W.H.O standards							
NESREA Effluent limits							

Table A.13 FC (MPN/100ml) characteristics of the treatment plant from May-Sept., 2015

Period	**Component of the treatment plant**				**Removal efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	920.00	-	650.00	130.00	85.87	20.00	20.00
Week 2	700.00	-	500.00	120.00	82.86	30.00	25.00
Week 3	750.00	-	400.00	80.00	89.33	50.00	51.00
Week 4	800.00	-	150.00	20.00	97.50	40.00	47.00
Week 5	850.00	-	100.00	30.00	96.47	50.00	50.00
Week 6	700.00	-	200.00	20.00	71.43	40.00	35.00
Week 7	950.00	-	22.00	10.00	98.94	16.00	15.00
Week 8	900.00	-	25.00	9.00	99.00	9.00	8.00
Week 9	980.00	-	35.00	15.00	98.46	11.00	15.00
Week 10	800.00	-	53.00	12.00	98.50	17.00	14.00
Week 11	782.00	-	44.00	8.00	98.97	16.00	13.00
Week 12	685.00	-	61.00	5.00	99.27	12.00	11.00
Mean	818.00	-	187.00	38.00	93.00	26.00	25.00
S.D	101.77	-	212.70	45.21	8.90	15.34	16.12
SKEW	0.22	-	1.36	1.46	-1.59	0.60	0.76
W.H.O standards							
NESREA Effluent limits							

Table A.14 pH characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Percentage change in Efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	7.24	6.96	6.85	6.99	3.45	7.35	7.25
Week 2	7.35	7.00	6.90	6.95	5.44	7.34	7.15
Week 3	7.46	6.98	7.00	6.85	8.18	7.30	7.10
Week 4	7.36	6.99	7.01	6.96	5.43	7.25	7.15
Week 5	7.40	6.79	6.99	6.90	6.76	7.30	7.25
Week 6	7.29	7.00	7.02	7.02	3.70	7.25	7.20
Week 7	7.20	7.00	6.98	7.15	0.69	7.16	7.15
Week 8	7.15	6.95	7.00	7.00	2.09	7.15	7.15
Week 9	7.15	7.10	6.98	7.16	0.13	7.20	7.16
Week 10	7.20	7.10	7.00	7.15	0.69	7.25	7.15
Week 11	7.16	7.15	7.10	7.20	0.55	7.00	7.15
Week 12	7.16	7.00	7.10	7.15	0.13	7.20	7.16
Mean	7.26	7.00	6.99	7.04	3.10	7.22	7.16
S.D	0.11	0.09	0.07	0.12	2.81	0.10	0.04
SKEW	0.63	-0.66	-0.46	-0.08	0.52	-1.08	0.99
W.H.O standards				6.5-8.5			
NESREA Effluent limits				6.9			

Table A.15 EC (μS/cm) characteristics of the treatment plant from May-September, 2015

Period	**Component of the treatment plant**				**Removal efficiency (%)**	**On river WUPA**	
	SCP 1	SCP 7C-2	SCP 8C-3	SCP 4		SP 1	SP 2
Week 1	315.00	312.00	298.70	225.50	0.95	215.00	230.60
Week 2	316.40	310.50	305.00	230.70	1.86	205.70	260.80
Week 3	290.00	295.00	286.60	175.68	1.69	190.00	202.00
Week 4	291.06	276.60	295.00	225.00	4.96	210.00	198.00
Week 5	310.50	312.63	300.65	235.81	0.68	211.20	205.60
Week 6	314.90	282.15	293.35	240.20	10.40	265.15	260.00
Week 7	220.00	215.00	202.00	190.00	13.63	220.00	225.00
Week 8	222.00	220.00	205.00	189.00	14.86	215.00	250.00
Week 9	225.00	222.00	210.00	195.00	13.33	208.00	212.00
Week 10	311.00	285.00	260.00	205.00	34.08	212.00	198.00
Week 11	258.00	236.00	224.00	200.00	22.48	220.00	210.00
Week 12	210.00	205.00	200.00	195.00	7.14	210.00	215.00
Mean	273.65	264.32	256.69	208.90	10.50	215.17	222.25
S.D	43.45	41.65	44.59	21.43	10.08	17.58	23.22
SKEW	-0.45	-0.23	-0.27	0.13	1.23	2.16	0.77
W.H.O standards				1250			
NESREA Effluent limits				600			

APPENDIX B

Table B.1 Average biochemical oxygen demand (BOD_5) characteristics of the treatment plant from January-December, 2014

Period	**Influent (mg/l)**	**Effluent (mg/l)**	**Removal Efficiency (%)**
January	153.00	18.40	87.97
February	210.00	21.00	90.00
March	250.00	18.00	92.80
April	267.00	22.600	91.53
May	160.00	11.70	92.68
June	106.00	9.40	91.13
July	85.00	15.80	81.41
August	70.00	7.67	89.04
September	69.00	10.80	84.34
October	70.00	13.40	80.85
November	106.00	18.00	83.01
December	160.00	17.80	88.87

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

Table B.2 Average chemical oxygen demand (COD) characteristics of the treatment plant from January-December, 2014

Period	**Influent (mg/l)**	**Effluent (mg/l)**	**Removal Efficiency (%)**
January	340.00	48.00	85.88
February	500.00	53.30	89.34
March	560.00	100.00	82.14
April	540.00	104.00	80.74
May	350.00	99.00	71.71
June	250.00	77.00	69.20
July	200.00	59.50	70.25
August	200.00	43.50	78.25
September	150.00	36.00	76.00
October	200.00	51.70	74.15
November	240.00	47.00	80.41
December	350.00	54.00	84.57

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

Table B.5 Average total suspended solid (TSS) characteristics of the treatment plant from January-December, 2014

Period	Influent (mg/l)	Effluent (mg/l)	Removal Efficiency (%)
January	126.00	15.00	88.09
February	209.00	17.00	91.86
March	299.00	15.00	94.98
April	240.00	18.80	92.16
May	120.00	14.80	87.66
June	116.00	17.60	84.82
July	120.00	26.70	77.75
August	110	23.3	78.81
September	100.00	19.60	80.40
October	70.00	18.70	73.28
November	96.00	21.60	77.50
December	126.00	17.20	86.34

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

Table B.6 Average dissolved oxygen (DO) characteristics of the treatment plant from January-December, 2014

Period	Influent (mg/l)	Effluent (mg/l)	Increase in Efficiency (%)
January	3.10	7.70	59.74
February	4.10	7.60	46.052
March	3.90	-	
April	3.50	7.30	52.05
May	4.50	7.50	40.00
June	55.00	-	
July	3.50	7.50	53.33
August	5.10	8.00	36.25
September	-	-	-
October	5.10	7.50	32.00
November	2.10	-	
December	2.00	7.00	71.42

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

Table B.7 Average nitrate as nitrogen (NO_3-N) characteristics of the treatment plant from January-December, 2014

Period	Influent (mg/l)	Effluent (mg/l)	Increase in Efficiency (%)
January	1.50	6.30	76.19
February	1.70	5.10	66.66
March	2.00	8.90	77.52
April	0.90	7.80	88.46
May	1.80	7.90	77.21
June	0.90	4.10	78.04
July	1.52	5.00	69.60
August	1.31	2.40	45.41
September	1.00	5.00	80.00
October	1.70	6.90	75.36
November	2.00	6.60	69.69
December	1.90	7.90	75.94

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

Table B.8 Average nitrite as nitrogen (NO_2-N) characteristics of the treatment plant from January-December, 2014

Period	Influent (mg/l)	Effluent (mg/l)	Removal Efficiency (%)
January	0.41	0.17	58.53
February	0.31	0.09	70.96
March	0.44	0.11	75.00
April	0.19	0.13	31.57
May	0.44	0.11	75.00
June	0.55	0.40	27.27
July	0.23	0.09	60.86
August	0.41	0.07	82.92
September	0.37	0.11	70.27
October	0.41	0.12	70.73
November	0.51	0.14	72.54
December	0.52	0.30	42.30

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

Table B.9 Average ammonium as nitrogen (NH_4-N) characteristics of the treatment plant from January-December, 2014

Period	Influent (mg/l)	Effluent (mg/l)	Increase in Efficiency (%)
January	0.70	4.10	82.92
February	1.20	6.40	81.25
March	2.90	6.90	57.97
April	0.75	1.50	50.00
May	0.95	4.60	79.34
June	0.52	12.50	95.84
July	0.49	4.10	88.04
August	2.00	1.45	27.50
September	1.40	1.20	14.28
October	1.50	8.00	81.25
November	1.40	2.70	48.14
December	1.90	6.90	72.46

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

Table B.10 Average chloride (CL^-) characteristics of the treatment plant from January-December, 2014

Period	Influent (mg/l)	Effluent (mg/l)	Removal/increase in Efficiency (%)
January	37.00	44.50	16.85
February	27.00	53.67	49.69
March	26.00	21.10	18.84
April	37.00	31.00	16.21
May	22.00	19.50	11.36
June	25.00	20.40	18.40
July	17.00	14.80	12.94
August	19.00	18.00	5.26
September	22.00	18.80	14.54
October	23.00	20.80	9.56
November	32.00	26.00	18.75
December	47.00	43.00	8.51

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

Table B.11 Average sulfate (SO_4^-) characteristics of the treatment plant from January-December, 2014

Period	Influent (mg/l)	Effluent (mg/l)	Removal/Increase Efficiency (%)
January	55.00	54.30	1.27
February	60.00	58.30	2.83
March	70.00	60.00	14.28
April	50.00	52.50	4.76
May	55.00	60.00	8.33
June	55.00	60.00	8.33
July	60.00	75.00	20.00
August	52.00	60.00	13.33
September	53.00	70.00	24.28
October	54.00	55.00	1.81
November	60.00	60.00	0.00
December	51.00	60.00	15.00

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

Table B.13 Average faecal coliform (FC) characteristics of the treatment plant from January-December, 2014

Period	Influent (MPN/100ml)	Effluent (MPN/100ml)	Removal Efficiency (%)
January	920.00	70.00	92.39
February	1600.00	45.00	46.05
March	1600.00	0.00	100.00
April	920.00	29.00	96.84
May	1600.00	0.00	100.00
June	920.00	0.00	100.00
July	1600.00	0.00	100.00
August	920.00	6.00	99.34
September	1600.00	25.00	98.43
October	1600.00	133.00	91.68
November	920.00	10.00	98.91
December	920.00	0.00	100.00

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

Table B.14 Average pH characteristics of the treatment plant from January-December, 2014

Period	**Influent**	**Effluent**	**Percentage change in Efficiency (%)**
January	7.30	7.07	3.15
February	7.10	7.08	0.28
March	7.20	6.94	3.61
April	7.29	7.06	3.15
May	7.30	7.03	3.69
June	7.25	7.10	2.06
July	7.30	7.20	1.36
August	7.15	7.15	0.0
September	7.16	7.16	0.00
October	7.25	7.10	2.06
November	7.20	7.11	1.25
December	7.10	7.15	0.69

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

Table B.15 Average electrical conductivity (EC) characteristics of the treatment plant from January-December, 2014

Period	**Influent (µS/cm)**	**Effluent (µS/cm)**	**Removal Efficiency (%)**
January	310.00	249.30	19.58
February	291.00	262.30	9.86
March	255.00	261.00	2.29
April	310.00	279.40	9.87
May	321.00	249.00	22.42
June	271.00	233.00	14.02
July	285.00	213.70	25.01
August	222.00	192.30	13.37
September	215.00	192.20	10.60
October	286.00	19.20	93.28
November	258.00	207.00	19.76
December	247.00	276.00	10.50

Source: Office of the Quality Control Officer WUPA Sewage Treatment Plant, Abuja

APPENDIX C

Plate I: The researcher sampling at the influent (SCP 1)

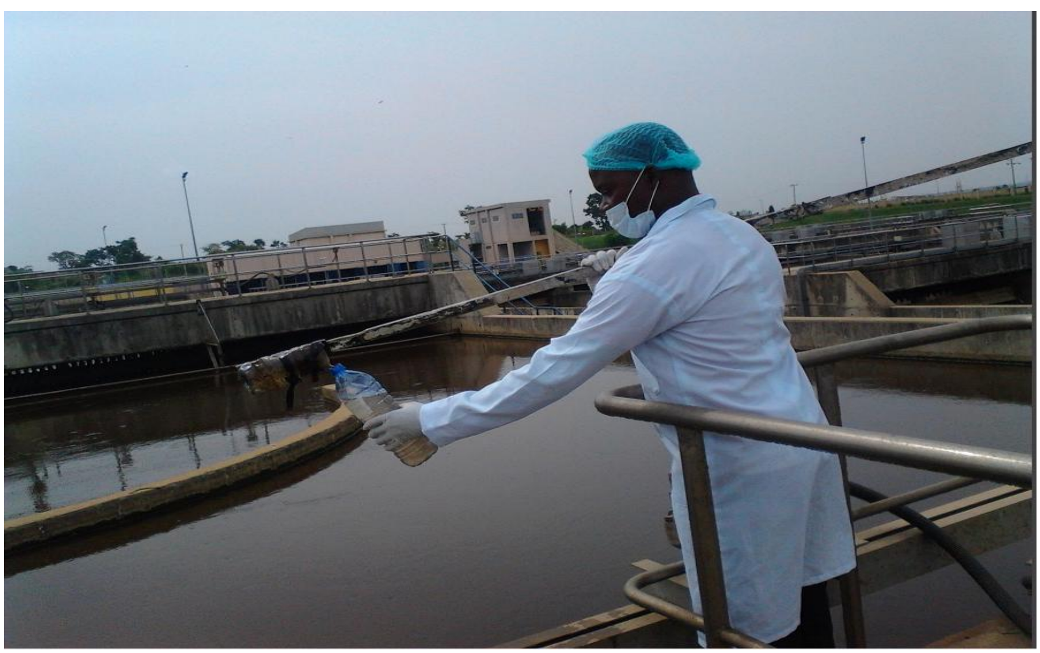

Plate II: The researcher sampling at the Aeration Tank (SCP 7C-2)

Plate III: The researcher sampling at the Sedimentation Tank (SCP 8C-3)

Plate IV: The researcher sampling at the Effluent Outlet (SCP 4)

Plate V: The researcher sampling at 10m upstream of River WUPA

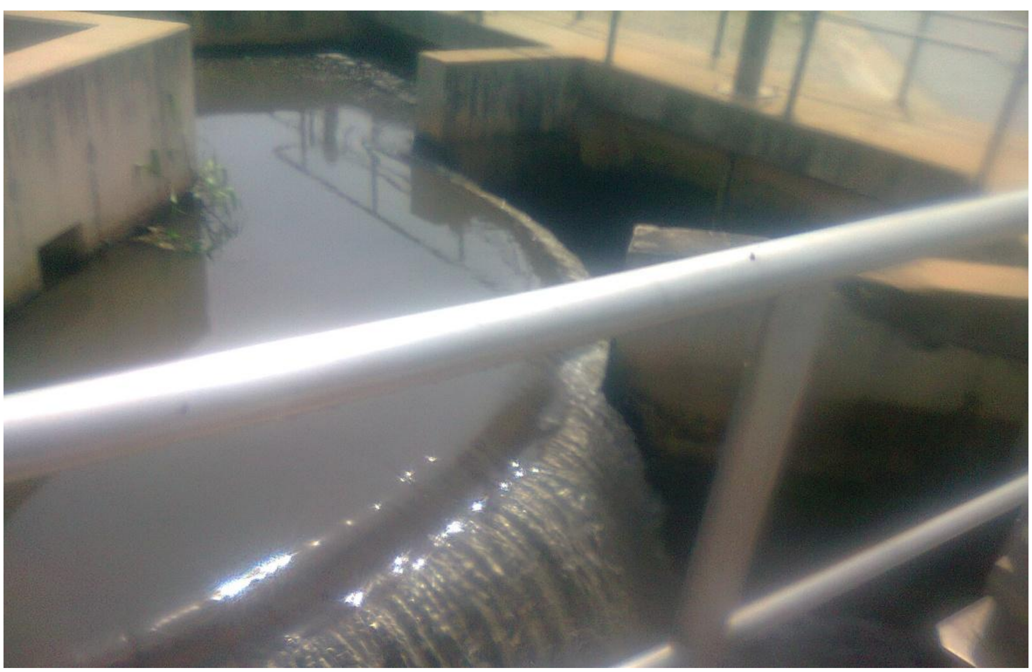

Plate VI: The sedimentation basin