POTENTIALS OF PALM KERNEL SHELL-BASED FIELD SCALE HORIZONTAL SUBSURFACE FLOW CONSTRUCTED WETLAND FOR ON-SITE BIO-REMEDIATION OF SLAUGHTERHOUSE EFFLUENT

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REG NO: 2013357001P

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE AWARD OF DEGREE OF DOCTOR OF PHILOSOPHY (PHD)

IN THE DEPARTMENT OF AGRICULTURAL AND BIORESOURCES ENGINEERING (ENVIRONMENTAL ENGINEERING OPTION)

FACULTY OF ENGINEERING,

NNAMDI AZIKIWE UNIVERSITY, AWKA

NOVEMBER, 2019

CERTIFICATION

I, **Okoye, Nelson Mbanefo** declare that this work was carried out by me under the supervision of Engr. Prof. C. N. Madubuike and Engr. Prof. E. I. U. Nwuba and it has not been submitted in part or full for any other diploma or degree of this or any other University.

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Okoye Nelson Mbanefo	**Date**

APPROVAL

This is to certify that Okoye, Nelson Mbanefo a postgraduate student of the Department of Agricultural and Bioresources Engineering, Faculty of Engineering with registration number 2013357001P has satisfactorily completed the requirements for the Degree of Doctor of Philosophy in Agricultural and Bioresources Engineering (Environmental Engineering Option). The work embodied in this thesis is original and has not been submitted in part or full for any other diploma or degree of this or any other University.

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DEDICATION

This work is dedicated to God Almighty, for thus far He has led me.

ACKNOWLEDGEMENTS

As I finalise this work, it is a pleasure for me to be able to thank some of the people who have taken an active part in this final achievement.

First and foremost, I thank the Almighty God for His infinite mercies, guidance, protection, provision and love over me and my family all these years. Thank you LORD. I wish to express my sincere appreciation to my Supervisors, Engr. Prof. C. N. Madubuike and Engr. Prof. E. I. U. Nwuba, who despite their very tight schedules found time to guide and supervise me. For providing endless assurance and encouragement, I highly appreciate the immense contributions of Dr L. C. Orakwe, the Head of Department, Agricultural and Bioresources Engineering, Nnamdi Azikiwe University. His very valuable advice helped to guide me through this work.

My big thanks go to Engr. Prof. G. O. Chukwuma and Dr. Nwajinka and to all the staff of Department of Agricultural and Bioresources Engineering for their immense contribution to this work. I also want to acknowledge the owners and operators of the Agulu slaughterhouse, especially Mr Paul Okonkwo and Mr Kenneth Ibe for making the facility available for this work.

My inspiration partly came from my wife, Uche and sons, Ryan and Richard. They believed that I could rise to the occasion the research posed and assisted in prayers. I am grateful to God for their love. I also appreciate my cousin Nonso Obiekezie for assisting with field sampling and data compilation. I thank my parents who have trained me to this level. Not forgetting my brother and sister, Dr Cliff and Juliet for their moral and financial support.

ABSTRACT

Substrates play an important role a Constructed wetland (CW). There has been a growing interest in the use of non-conventional materials as CW substrates. Palm kernel shell have been reported to have the capacity to serve as a wetland substrate. Its use in CWs will lead to a beneficial reuse of the by-product. Therefore, the aim of the study is to explore the potential of a palm kernel shell based field-scale horizontal subsurface flow CW for on-site bio-remediation of slaughterhouse effluent. To this end, several research tools including pilot and field-scale studies and computational fluid dynamics modeling were employed. Preliminary studies were conducted with a view to ascertaining the growth and treatment performance of three macrophytes, and the suitability of palm kernel shell as a CW substrate. Six pilot horizontal subsurface flow CW cells were built and planted with three macrophytes *Typha latifolia, Thalia geniculata* and *Colocasia esculenta.* Four of the cells were filled with gravel, while two were filled with palm kernel shell. Influent and effluent wastewater samples were collected and evaluated for key physicochemical parameters. To estimate *k-C** design model constants, three horizontal subsurface flow CW columns were built and the model constants obtained by fitting the model predictions to the measured concentrations in the column. A palm kernel shell based field-scale horizontal subsurface flow CW was constructed and monitored for key physicochemical parameters. The hydrodynamic behaviour of the field-scale CW was evaluated using tracer test. Also two Dimensional (2D) computational fluid dynamic modeling using finite element-based commercial software COMSOL Multiphysics 5.3a was employed to further evaluate the hydrodynamic behaviour of the system, and to simulate the influence of different hypothetical configurations to optimize residence time. The preliminary study revealed that *Thalia Geniculata* was the most suitable macrophyte specie for CW with palm kernel shell. It also revealed that palm kernel shell had a treatment efficiencies of 72.81% for BOD; 89.87% for TSS; 39.42% for NH_4-N; 60.79% for NO_3-N and 42.52% for PO_4^{3-} comparable to values of 75.42% for BOD; 88.18% for TSS; 41.33% for NH_4-N; 55.86% for NO_3-N and 44.73% for PO_4^{3-} obtained for gravel. The palm kernel shell based field-scale horizontal subsurface flow CW significantly reduced pollutant concentration of the slaughterhouse effluent, with average removal rates of 81.07% for BOD, 82.12% for TSS, 46.03% for NH_4-N, 38.13% for NO_3-N and 40.92% for PO_4^{3-}. The hydrodynamic evaluation showed that water fluxes were not homogeneous, but that the system had a good hydraulic efficiency. The computational fluid dynamic modeled tracer response curve showed good agreement with the experimental results, with a correlation coefficient of 0.99. It also revealed that vegetation layout within the wetland was the most effective modification for improving the hydrodynamics, thus should be given adequate consideration during the design phase. The study concluded that palm kernel shell based field-scale horizontal subsurface flow CW improves effluent quality, with removal rates comparable to that of conventional wastewater treatment systems, thus should be used protect sensitive water bodies that receive slaughterhouse effluent. It has provided rigorous field data and information to support its implementation. The study recommends long term (5 to 10 years) performance evaluation of palm kernel shell substrate, with a view to determining the magnitude of its lifespan.

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LIST OF ABBREVIATIONS AND NOMENCLATURE

- A = Area [m3]
- B.I = Biodegradability Index
- BOD = Biological Oxygen Demand [mg/L]
- C = Concentration [mg/L]
- ^{o}C = Degrees Celsius
- C* = Background Concentration [mg/L]
- CFD = Computational Fluid Dynamics
- COD = Chemical Oxygen Demand [mg/L]
- CSTR = Completely Stirred Tank Reactor
- CW = Constructed Wetland
- d = Day
- DTD = Detention Time Distribution
- DM = Dispersion Model
- DO = Dissolved Oxygen
- EC = Electrical Conductivity
- FEPA = Federal Environmental Protection Agency
- FWS = Free Water Surface
- g = Gram
- h = Height or Depth [m]
- HDPE = High Density Polyethylene
- HLR = Hydraulic Loading Rate [m/d]
- HRT = Hydraulic Retention Time [d]
- HSSF = Horizontal Subsurface Flow
- ICW = Integrated constructed wetland
- k_{20} = Removal Rate Constant at 20 oC [g/m2/d or m/d]
- k_T = Removal Rate Constant at Temperature "T" [g/m2/d or m/d]
- l = length [m]
- L = litr es
- LDP = Limiting design parameter
- LSOM = Least Squares Optimization Method
- N = Number of tanks in series

- NESREA = National Environmental Standards and Regulatory Enforcement Agency
- NH_4-N = Ammonia Nitrogen
- NO_3-N = Nitrate Nitrogen
- NO_2-N = Nitrite Nitrogen
- MOM = Method of Moments
- MRA = Multiple Regression Analysis
- MRR = Mass Removal Rate
- PO_4^{3-} = Ortho-Phosphate
- P = Number of apparent tanks in series
- PKS = Palm Kernel Shell
- PFD = Plug Flow with Dispersion
- PFR = Plug Flow Reactor
- PVC = Polyvinyl Chloride
- Q = Influent Hybrid-SSFCW Wastewater Rate [m3/d]
- R^2 = Coefficient of Determination
- RBC = Rotating Biological Contractor
- SSF = Subsurface Flow
- SSQE = Sum of Squares Error
- TBC = Tracer Breakthrough Curve
- TDS = Total Dissolved Solids
- TIS = Tanks-In-Series Type Kinetic Model
- TSS = Total Suspended Solids [mg/L]
- VW = Vertical Subsurface Flow Constructed Wetland
- Δt = Time Interval [d or hr]
- ε = Effective Porosity
- θ = Modified Arrhenius Temperature Factor
- τ = Hydraulic Retention Time [d or hr]

CHAPTER ONE

INTRODUCTION

1.1 Background Information

The need to protect water bodies, which are essential components of the natural ecosystem, from all sources of pollution, have become very necessary, because human existence is dependent on their sustainable utilization. Unfortunately, these essential renewable resources, which are freely available for various important needs such as domestic, agricultural and recreational uses, are increasingly threatened by different pollutants, including organic matter, nutrients and heavy metals.

Although natural process like climate and geology can negatively influence the intergrity of surface and groundwater, anthropogenic activities constitute the main sources of contamination. In Nigeria, contamination of large water bodies by discharges from industrial and agricultural operations has been reported (FEPA, 1995, World Bank, 1998). The ultimate goal of wastewater remediation is to protect humans and the ecosystem by protecting both surface and groundwater supplies and preventing the spread of waterborne diseases. Recently, however, industrial and agricultural expansions, combined with poor enforcement of already existing environmental legislations, have resulted to widespread environment deterioration.

A major cause of environmental pollution in Nigeria is contamination of water bodies by the meat-processing industry. Because of the extreme negative impacts of slaughterhouse effluent on humans and the environment, the pollution burden of slaughterhouses is now considered a matter great scientific interest (Sangodoyin and Agbawe, 1992; World Bank, 1998; Osibanjo and Adie, 2007). The meat processing industry uses very large amounts of water and therefore discharges quite a significant volume of wastewater. For example, 1.5 to $10m^3$ per ton is required for pig processing, while for beef and poultry, it ranges from 2.5 to $40m^3$

and 6 to $30m^3$ repectively (World Bank, 1998). The benchmark set by British Environment Agency for water consumption is 0.7 to $1.0m^3$ per cattle slaughtered (Environment Agency, 2008). This means that a facility that slaugahters 100 cattle per week will produce at least 70 m^3 (70,000 liters) of wastewater each week, for which treatment is required. In a developing country such as Nigeria, the amount of wastewater generated may not be as great as that generated in the developed world due to a shortage of potable water, which is evident in most slaughterhouses, but the volumes discharged daily are enough to contaminate the receiving environment.

Meat processing industry wastewater is a combination of water used for cleaning the slaughtered animal carcass, the slaughterhouse, personnel and equipment (Coker *et al.*, 2001). The wastewater from slaughterhouses is characterized by nitrogen-rich biodegradable materials, suspended and dissolved solids, fat residues, blood, hair, intestinal contents, detergents and skin residues (Ojo 2014). Coker *et al*., (2001) stated that the key constituents of slaughterhouse wastewater are blood and gut contents particularly particles from semi-digested and undigested feeds.

In developing countries, slaughterhouses are usually sited in areas that have access to water bodies so as to guarantee regular supply of water (Akan *et al.,* 2010). Abattoir effluents are known to contaminate both surface and groundwater, with very serious negative health implications (Ubwa *et al*., 2013; Atuanya *et al*., 2012; Adeyemi-Ale, 2014; Adegbola and Adewoye, 2012; Adesomoye *et al*., 2006). If contaminations of this magnitude continue without any form of mitigation, then long lasting ecological damage is unavoidable. Thus to ensure environmental sustainability and the protection of users of these fairly common and very indispensable natural elements, it is therefore necessary that wastewater from slaughterhouses is treated before it is discharged.

Conventional treatment systems exist for different types of wastewater treatment, such as trickling filters, rotating biological contactors, stabilization ponds etc., but most have been unsustainable in Nigeria (Badejo *et al*., 2012). According to Komolafe *et al*., (2013) they are associated with very high levels of automation, and factors such as poor maintenance, insufficient high-skilled personnel to operate modern equipment, inadequate power supply for problem-free operation of sensitive systems, lack of security, and high installation and maintenance costs are some of the obstacles that pose serious threat to its success in Nigeria. Also the performance records of some of these conventional treatment systems were not very encouraging (Kadlec and Hammer, 1988). Therefore, the serious sanitation challenges in the country require the application of a cheap, effective and sustainable treatment technology.

There has been an increased interest in the use of decentralized treatment technologies, especially in developing countries where centralized systems have been difficult to implement due to a number of factors. According to Mara (2004), the selection of wastewater treatment systems for developing countries should be based on the following criteria: low installation, operation and maintenance costs; should not be diffcult to run; should use little or no energy, prefareably natual energy sources; should also use little or no chemicals; should be efficient enough to produce effluents that meet or are close to the discharge standards; other factors such as low level of sludge production and minimal land intake are also crucial. CWs meet most of the above criteria and can therefore be a suitable wastewater treatment technology for developing countries.

CWs are treatment systems that utilize natural processes involving macrophytes, substrates and microorganisms to remove pollutants from wastewater. CWs rely on various physical, biological and chemical processes to improve wastewater and their reduction efficiency is dependent of key factores such as their design, operation and maintenance. CWs have been found to lower construction and maintenance costs and use relatively low-skilled labour compared to other types of treatment systems (USEPA, 1999). CWs also been reported to be efficient in organic, pathogen and nutrient reduction, as well as other contaminants (Konnerup *et al.*, 2009). While the primary focus of conventional systems is large scale treatment of wastewater in urban areas, CW systems are considered suitable for decentralized wastewater treatment in low density areas. It is known that the warm climatic throughout the year in the tropics stimulates high plant productivity and also creates favourable conditions required by biological communities to grow and break down harmful substances. However, limited studies have been conducted on the performance of field-scale CWs in the humid tropics, particularly for the treatment of abattoir wastewater.

The historical background of CW utilization for the remediation of wastewater started with the study conducted by K. Seidel in Germany in the early 1950s (Ohio EPA, 2007, Dhulap and Patil, 2014, Kadlec and Wallace, 2009). Its use evolved over time and at different rates for a number of countries. Application in the US began in 1967 (Kadlec and Wallace, 2009), while it began in 1985 in the United Kingdom (Cooper and Green, 1995). CW research has evolved over the past half century into an interdisciplinary field with a wide range of motivations and perspectives, but the main focus has always been water quality improvement. Initially, its use was limited to domestic or municipal sewage treatment. Over the past three decades, however, significant increase in the use of CWs has been reported, with many more countries now employing CWs for the treatment of different waste streams such as for leachate treatment of waste dumps, agricultural and urban runoff, wastewater from food processing, chemical and other industries, as well as effluent from refineries (Vymazal, 2008, 2010, Kadlec and Wallace, 2009). CW has been identified as a viable option for wastewater treatment in many countries due to its environmentally friendly nature (Kadlec and Wallace, 2009).

While notable successes have been achieved with CWs in the developed world for the remediation of different types of effluents under different scenarios, the same cannot be said about their use in developing countries, even with the more appropriate climatic conditions. Denny (1997) described the spread of constructed wetland technology in developing countries as "depressingly slow". There seems to be a general reluctance to invest money and time in wastewater treatment systems such as CWs. This is particularly true for Nigeria, where the technology is not traditionally known and has not been introduced noticeably despite the attractive potential. Although there are a number of cases where CWs (mainly surface flow planted with water hyacinth, *Eichhornia crassipes spp.*) are used as secondary treatment units for wastewater in the country, the technology so far is not yet common for the same purpose and the few that are operational perform below the required standards (Adeniran *et al*., 2012). CWs have not yet received the deserved attention in the country, both from public and private institutions, as a viable option for pollution reduction. For example, there is not even one complete CW that has been built or operational for any form of wastewater treatment throughout Southeast Nigeria. Hence, the reason for this study is to explore the use of CWs for slaughterhouse effluent bio-remediation, particularly with Palm Kernel Shell (PKS). Nigeria presently occupies the fifth position amongst the oil palm producing nation, accounting for 1.5% of the global production (Izah and Ohimain, 2016). The total hectarage for oil palm in Nigeria was estimated at 3,053,974 hectares (FAO, 2018) and approximately 15 to 18 tonnes of palm kernel fruit is produced per hectare each year, with Palm Kernel Shell (PKS) making up approximately 64% of the mass (Okoroigwe *et al*., 2014).

1.2 Statement of Problem

The disposal of PKS has always been a challenge for oil palm producers. Burning to produce energy, which is the common practice, is against the principles of environmental conservation, and thus not eco-friendly. Therefore, there has been an increasing interest in the reuse of PKS for various purposes, such as using PKS as aggregate materials in light weight concrete structures and also as a material for activated carbon for wastewater treatment. However, these reuse options have still not significantly reduced the quantities of PKS that needs to be disposed of annually. Previous studies on the properties of PKS suggest they could be a suitable substrate in CWs. Substrates are an important component of CWs as they strongly influences the installation and maintenance costs, as well as purification capability of horizontal subsurface flow CW. Utilization of PKS for wastewater treatment in CWs will not only lead to a beneficial reuse of this potential resource, which is in accordance with the concept of sustainable development, but will also lead to a significant cost reduction. However, PKS as a wetland substrate has been scantly researched. Almost all the studies were conducted on bench scale systems and no study has evaluated PKS as a wetland substrate for slaughterhouse wastewater treatment. Thus there is need for field scale evaluation its treatment potentials. The most important controlling factor for various interconnected processes that occur within a HSSF CW is water movement patterns. With the conventional wetland configurations currently in use, short-circuting flows occur, thereby negatively influencing the hydraulic and treatment efficiencies of theses systems. Research on the hydraulic modeling and optimization of bed configuration and design is limited. Therefore, for PKS to be adopted as an alternative substrate for HSSF CWs, it is necessary to use simulation models to gain better insight into PKS bed functioning and hydraulic performance.

1.3 Aim and Objectives

The primary focus of this research is the on-site bio-remediation of slaughterhouse effluent using field-scale horizontal subsurface flow constructed wetland. The specific objectives are as follows:

1. To characterize wastewater from slaughterhouses in selected towns of Anambra State.
2. To Identify and compare the growth characteristics and the treatment response of appropriate locally available macrophytes that can be used in PKS based HSSF CWs.

3. To simulate the removal of pollutants from slaughterhouse wastewater using batch-operated HSSF CW columns in order to estimate design model parameters.
4. To build and evaluate the performance of an experimental field-scale PKS based HSSF CW for slaughterhouse effluent bio-remediation.
5. To model the PKS based HSSF CW hydraulics and optimize the system design.

1.4 Justification of the Study

Based on the situation already presented, the current research focuses on the performance of a HSSF CW, with the intention of addressing the following pertinent issues:

1. There is a paucity of data that on the quality and quantity of wastewater from slaughterhouses in Nigeria, which can help engineers to take the necessary procedures or precautions while trying to find solutions to the problems associated with slaughterhouse wastewater.
2. Comparative studies on the treatment potentials and survival of macrophyte species specific to the study area, in CW treating high-strength slaughterhouse effluent will minimize system failure.
3. Rigorous field data on the actual process performance of HSSF CW for slaughterhouse effluent under humid tropical environmental conditions is lacking, and this is crucial for informed design and decision making.
4. The hydraulic performance of existing constructed wetlands is often compromised by hydraulic problems. Therefore, the development of an appropriate simulation model, to reliably predict how various modifications of bed design and configurations might affect performance, will facilitate the design of efficient systems.

1.6 Scope of the Study

1. The purview of this work is the physicochemical evaluation of pilot and field-scale HSSF CWs for slaughterhouse wastewater treatment. Microbial parameters (Feacal and

Total Coliforms) were not monitored as several literatures have reported almost total removal (99.9%) by different wetland systems and also because of funding constraints.

2. The study only investigated the influent and effluent wastewater characteristics. Investigation of mechanisms for the removal of pollutants and influence of the properties of the substrate material were not carried out, as it was beyond the scope of this work.
3. Issues of constructed wetland clogging were not addressed as evaluation of bed clogging is a long term study, and thus could not be accommodated within the limited time frame for this study.
4. 2D CFD was used to model the velocity field and the residence time distribution for HSSF CW because of limited computing power that was necessary for 3D modelling.
5. Potential physics like evapotranspiration, solar radiation, prevailing temperature and humidity etc which influences processes that occur in the system were not included in the modeling process because of the lack of a controlled environment that was necessary for the collection of such data.

CHAPTER TWO

LITERATURE REVIEW

2.1 State of Industrial Wastewater Management in Nigeria

Surface and underground water sources potential of Nigeria is very large and is valued at 267 and 51 billion m^3 respectively (IMF, 2005). Nigeria's population in 2016 was estimated at about 193 million (NBS, 2018). The growing population of the country has led to an intensive increase in the exploitation of natural resources due to expansion of urban centers. As a result, more waste streams reach water bodies. This has also resulted to significant pollution of aquatic environment and a compromise of ecosystem integrity. Industrial (chemical, textiles, brewery), agricultural, pharmaceuticals etc are amongst the major contributors to environmental contamination in Nigeria, with the oil and gas industry posing the greatest threat for the aquatic environment (Adedeji and Adetunji, 2011).

It had been previously reported that out of about 200 industries in Lagos only 18% treated their effluent before discharge into nearby surface waters (FEPA, 1995). Although recent statistics are not available, there have been little changes compared to the 1995 ssituation. Surface waters continue to be polluted with little or no efforts made by the agencies saddled with such responsibility to preventing it. Pharmaceutical industries, paint and fertilizer plants, slaughterhouses, cement factories and steel and metal plants were amongst the major contributors hazardous waste discharge between 1988 and 1991 (FEPA, 1995).

The environmental impact of industrial effluent discharges in Nigeria has been extensively studied. Adekunle (2009) investigated its impact on well water quality and reported the following ranges: 1.5 and 250NTU for turbidity; 211 to 2519Pt-Co for colour; 161 to 731 μs/cm for conductivity; 6.9 to 7.3 for pH and 6 to 9mg/l for dissolved oxygen. Values of other contaminants such as total suspended and dissolved solids, calcium, magnesium were also high.

The values recorded for total bacteria counts (1200 - 1375 cfu/ml) also revealed water that was heavily polluted.

Udiba *et al*., (2013) studied mining impact on groundwater quality in Dareta Village, Zamfara. The mean level of the examined parameters was temperature (29.08±0.22oC), pH (6.34±0.26), electrical conductivity (370.83±179.16µs/cm), total dissolved solids (174.33±100.02mg/l), Nickel (0.06±0.05mg/l), Chromium (0.17±0.07mg/l), Manganese (0.14±0.10mg/l) and Magnesium (2.48±0.27mg/l). They concluded that temperature and pH do not fall within the limits set by the WHO and the Nigerian drinking water quality standard (NSDWQ) and also that the water samples were severely contaminated by Nickel, Chromium and Magnesium.

The disposal of waste in landfill sites is considered a means of reclamation of gullies and excavations in Nigeria. However, leaks from such landfill sites contribute to contamination of the environment. Akinbile and Yusoff (2011) assessed the environmental effects of landfill leachate on groundwater. The values of pH, turbidity and temperature varied from 5.7 to 6.8, 1.6 to 6.6NTU and 26.5 to 27.5°C respectively. Iron levels varied from 0.9 to 1.4mg/l, Nitrate from 30 to 61mg/l, Nitrite from 0.7 to 0.9mg/l and Calcium varied from 17 to 122mg/l. Heavy contaminations were also reported for heavy metals, with values of 0.3 to 2.3mg/l recorded for Zinc and 1.1 to 1.2mg/l recorded for Lead.

Water pollution has in general caused unpleasant health effects in Nigeria and the third world, especially if the water source serves for drinking purposes. In developing countries, a significant percentage of deaths are caused by the consumption of polluted water (Chikogu *et al*., 2012). Nearly 14,000 deaths in developing countries are attributed to the consumption of water contaminated by sewage (Owa, 2013). Among the water-borne diseases that are responsible for the loss of several million lives worldwide are typhoid, cholera and diarrhoea. Diarrhoea kills no less than 3 million people annually (especially children younger than five years); Typhoid kills

600,000 annually, while cholera claims an estimated 120,000 lives each year (Demena *et al*., 2003). These diseases occur primarily due to exposure to polluted water.

Studies have shown that intake of heavy metals can also lead to various health issues like kidney, liver and brain problems (Mudgal *et al*., 2010). One of the ways in which heavy metals are taken by people is through direct consumption of polluted water. An investigation by Ibeto and Okoye (2010) on 240 men, women and children in the state of Enugu noted that the concentration of Nickel, Manganese and Chromium in some of the blood samples was very high and exceeded the WHO allowable limit.

Various initiatives, laws and policies have been put in place and many international conventions have been ratified and domesticated by the federal government to mitigate environmental pollution (Ajayi, 2011). Key amongst these is the establishment of the Federal Environmental Protection Agency (FEPA). It was set up by Decree 58 of 30th December 1988 (FGN, 1988). It was later replaced in 2007 by the National Environmental Standards and Regulation Enforcement Agency (NESREA), with the mandate to monitor and enforce compliance to legislations on pollution prevention. The agency also has the authority to draw up and revise regulations in the area of air and water pollution control (Environmental Law Research Institutes). All efforts by various arms of government in Nigeria to ensure minimized environmental problems through the very many environmental legislations have not yielded the desired results, as the country continues to grapple with serious environmental challenges (Amokaye, 2012). These legislations are poorly enforced, and wastes are still randomly disposed of thereby polluting the environment.

2.2 Slaughterhouse Wastewater

A slaughterhouse is defined as any approved premise for inspection, hygienic slaughtering of animals and preservation for human consumption (Akinyeye *et al*., 2012). In Nigeria, siting of most slaughterhouses is not regulated; with access to surface water remain the major consideration in most cases. They generate large volumes of wastewater and significant quantities of solid waste. Wastewater from slaughterhouses poses a great environment and health challenge (Ubwa *et al*., 2013; Sangodoyin and Agbawe, 1992).

It is estimated that 35% of the weight of the slaughtered animal is converted to waste (World Bank, 1998). However, Verheijen *et al*. (1996) stated that approximately 5.5kg of manure 100kg undigested feed are generated per 1000kg of carcass weight, and that carcass weight is reduced after slaughtering from 400 to 200kg for a cow, with a third of the weight lost after bone and fat extraction. Thus, 35% of the animal live weight gives edible meat, while 65% are wastes.

Several factors affects the strength of the effluent from slaughterhouses, including blood collection, type of slaughtered animals and water usage (Tritt and Schuchardt, 1992). When adequate volumes of water are not used, the resulting effluent becomes very strong due to limited dilution. Blood retention in the slaughterhouse is considered critical in reducing the strength of effluents.

2.2.1 Pollution Potentials of Slaughterhouse Effluent

Poor management of slaughterhouses have been identified as the major contributing factor to the continued risk they pose to the environment and public health in Nigeria. The slaughterhouse's activities produce large volumes of wastewater with a characteristic high content of organic substances, suspended and dissolved solids and fats (Akinro *et al*., 2009). Tritt and Schuchardt (1992) stated that blood has a COD value of 375,000mg/l. COD, BOD and TSS values of 22,000 - 27,500mg/l; 10,800 - 14,600mg/l and 1,280 - 1,500mg/l respectively have also been reported (Sunder and Satyanarayan, 2013). In a study carried out by Mittal (2004) in Canada, values of 2,333 - 8,620mg/l for TS; 736 - 2.099mg/l for TSS; as well as 6 and 2.3mg/l for Nitrogen and Phosphorus respectively were recorded for abattoir wash down.

The effects of untreated wastewater from slaughterhouses on surface water in Nigeria has been extensively studied. Ubwa *et al*., (2013) assessed the surface water pollution status around Gboko abattoir and believed that activities in the slaughterhouse contributed to the pollution of the waters in the area. The values of TDS, TSS, DO, BOD and PO_4^3 did not meet the set limits for discharge. Atuanya *et al*., (2012) have observed consequential difference between the downstream and upstream water quality parameters, which clearly indicated the negative consequences of the discharge of wastewater from the slaughterhouse on the Ikpoba River in Benin City. Ojo (2014) assessed the environmental impact of wastewater from the municipal slaughterhouse oko-oba in Agege, Lagos. Physicochemical and bacteriological properties of water samples from the stream in which wastewater was discharged and groundwater samples around the area were analyzed. The values obtained for groundwater and stream water were significantly high. Adeyemi-Ale (2014) in their evaluation of the impact of wastewater from the slaughterhouse on the quality of the stream water, noted that the average values of physicochemical parameters were significantly higher than the recommended limits for discharge. Magaji and Chup (2012) in their own study reported that most of the analyzed properties of the stream water were still below the national and internationally accepted limits, but warned of the potential danger of continuous discharge of these waste materials in the stream.

The negative impact of wastewater from slaughterhouses on groundwater reserves has also been reported. Sangodoyin and Agbawe (1992) in their investigation of the influence of abattoirs effluent on groundwater in Ibadan reported high pollution strength of slaughterhouse effluents and stated that the quality of the groundwater at about 250 meters from the slaughterhouse was unsatisfactory as a source of water for drinking purposes. Adegbola and Adewoye (2012) in their investigation into the contamination of groundwater by Atenda abattoir found that the value of the total count of coliforms from the water samples exceeded the recommended range for drinking water. They concluded that the presence of a large number of coliforms in the water system of the immediate vicinity of the Atenda slaughterhouse was responsible for the observed cases of diseases outbreaks in the area. Other studies show that the effluents from slaughterhouses negatively affect the soil. Rabah and Oyeleke (2010) assessed the microbiological and physicochemical contamination of the soil by the effluent from slaughterhouses in Sokoto Metropolis and concluded that there were high counts and varieties of pathogenic microorganisms in the contaminated soil samples.

Slaughterhouse wastewater contains, among other pollutants, nutrients such as Nitrogen and Phosphorus, excess of which have been found to cause algal blooms in aquatic ecosystems and related eutrophication problems and generally changed ecosystem performance. The pollution load of pathogens in the wastewater of slaughterhouses is very variable, but can potentially reach high and harmful concentrations. Various microorganisms have been identified in slaughterhouse effluent such as S*taphylococcus spps, Escherichia coli spps, Salmonella spps* etc, (Coker *et al*., 2001; Adesomoye *et al*., 2006; Atuanya *et al*., 2012).

Poor management of the waste water from slaughterhouses and the subsequent removal, directly or indirectly, in water bodies predicts serious risks for the environment and health, both for aquatic life and for humans. Thus, in order to protect against the negative effects of waste water from the slaughterhouse, there is a need for adequate treatment before it reaches the receiving environments.

2.2.2 Slaughterhouse Wastewater Management in Nigeria

Wastewater management in Nigerian slaughterhouses is poor. The majority of slaughterhouses do not have basic waste and wastewater treatment and disposal facilities. For example, a study by Fadare and Afon (2010) reported that the storage and disposal practices of waste and wastewater in 80% of the slaughterhouses in Ile-Ife were not environmentally friendly.

Adesokan and Sulaimon (2014) in their research on poor management of slaughterhouse waste in Nigeria and its implications for reaching the millennium development goals reported that 74.4% of the 309 randomly selected slaughterhouses in Nigeria discharged their wastewater into the surrounding rivers.

Several research have investigated the environmental and health implications of untreated waste water from slaughterhouses in Nigeria (Adesomoye *et al*., 2006; Atuanya *et al*., 2012, Ubwa *et al*., 2013; Adeyemi- Ale, 2014; Ojo, 2014), but there are only a few research activities on the development and evaluation of efficient and cheap solutions for the management of wastewater from slaughterhouses in Nigeria. For example, Nwabanne and Obi (2017) employed electrocoagulation using iron electrodes for the remediation of abattoir wastewater and concluded that the use of Fe-Fe electrodes is an effective method. However, the replication of this technique on a field-scale is rare in the country.

As already noted, socio-economic and infrastructural challenges in the country have not allowed the implementation of some of these advanced treatment systems available in the developed world. Mijinyawa and Lawal (2008) assessed the treatment efficiency of a conventional poultry slaughterhouse wastewater treatment plant comprising of screening; primary clarification; chemical coagulation; chlorination; neutralization; sedimentation; carbon, sand and bag filtration; and UV light purification in Ibadan, Nigeria. The operating costs of the system were estimated at ₦69,493.63 per day. The majority of slaughterhouses in the country are small to medium sized and may as such not afford such high operating costs. Thus, CW has the potentials to be a viable alternative for the treatment of wastewater from slaughterhouses in Nigeria, particularly because of the associated low costs and ease of operation. The treatment of wastewater from slaughterhouses with CWs is however more complicated because of the high strength of slaughterhouse effluents compared to domestic wastewater, as shown in Table 2.1

Table 2.1 Characteristics of meat processing plant effluent and raw sewage

Parameter(mg/l)	Slaughterhouse Wastewater		Raw Sewage Water
	Cooper and Russell (1991)	Tritt (1992)	Badejo et al., (2012)
BOD	700-1,800	1,000-4,000	210-370
COD	1,000-3,000	1,000-6,000	420-450
TKN	70-180	250-700	-
NH3-N	5-50	50-100	10-30
TSS	200-1,200	-	167-231
Fat	100-900	-	-
Total Phosphorus	5-20	80-120	2-8.5

2.3 Wetlands

Areas with depth of ground water on the ground level or just below the ground level over a greater part of the year, resulting in changes in the chemical, physical and biological properties of the soil and which only allow plants that can adapt and survive severe flooding, are also known as wetlands (Kadlec and Wallace, 2009). Wetlands are divided into two, namely natural and CWs. While natural ones that exist through natural processes instead of anthropogenic influences, CWs are engineered systems (Vymazal, 2010). The United Nations Human Settlement Program defined CWs as a low-cost biological treatment systems designed to mimic the functions of natural wetland (UN-Habitat, 2008). They are a complex system made up of an integrated of water, macrophytes and microorganisms. They remove pollutants in wastewater through a combination of processes such as filtration, sedimentation, microbial processes, plant uptake, precipitation, adsorption etc (Kadlec and Knight, 1996). The areas of application of CWs have become very broad, varying from the secondary treatment of different types of wastewater, to the tertiary treatment and polishing of wastewaters treated in other conventional ways. Table 2.2 gives case studies of CW application for different types of wastewater

Table 2.2 Case studies of constructed wetland application

Wastewater Type	References
Refinery Wastewater	Ferro *et al.*, (2002); Litchfield *et al.*, (1989)
Dairy Effluent	Dipu *et al.*, (2010); Tanner *et al.*, (2005)
Domestic Wastewater	Adeniran *et al.*, (2014); Chang *et al.*, (2012)
Hospital Wastewater	Badejo *et al.*, (2012)
Abattoir Wastewater	Carreau *et al.*, (2012); Poggi-Varaldo *et al.*, (2002)
Stormwater Runoff	Pontier *et al.*, (2004); Revitt *et al.*, (2004)
Landfill Leachate	Wojciechowska and Obarsa-Pempkowiak (2008); Johnson *et al.*, (1999)

CWs are used instead of or alongside other wastewater treatment systems and are considered as a viable option because construction and maintenance costs are lower than traditional wastewater treatment plants. Some of these traditional methods are briefly described below:

- *Activated-sludge process*

In the activated-sludge system, wastewater is treated by an active mass of micro-organisms under aerobic conditions which are provided either by mechanical aeration or other natural diffusion processes.

- *Aerated lagoons*

The aerated lagoon, which is very much similar to the activated-sludge process in terms of microbial assemblages, uses a shallow basin of 1-4m depth. Wastewater is treated as it flows through the system.

- *Trickling filters*

This treatment system uses a permeable medium on which microorganisms grow and attach. Wastewater percolates through the slime layer formed in the system and treatment occurs in the process. Trickling filter is considered one of the most common biological treatment system for organic matter remediation from wastewater.

- *Rotating biological contactors*

This treatment system is similar to the trickling filters becaused it is also an attached-growth biological reactor. In a rotating biological contactor, large circular disks which are mounted on horizontal shafts rotate slowly, submerging the disks in wastewater, and microorganisms attached to the disks degraded organic matter in the process.

- *Stabilization ponds*

Stabilization ponds are very shallow wastewater ponds contained in an earthen basin, using a completely mixed biological process without solids return. Mixing may be either natural (wind, heat or fermentation) or induced (mechanical or diffused aeration).

2.3.1 Types and Configuration of Constructed Wetlands

There are two broad classifications of CWs. The first is based on the hydrology of the system (surface and subsurface flow) and the second is based on the type of vegetation used in the system (emergent, submerged, free-floating). Subsurface flow CWs are further classified into two based on the direction of flow (vertical or horizontal) (Vymazal, 2010).

The free water surface (FWS), also called surface flow wetland, is made of a shallow sealed basin that is lined to prevent seepage. Floating, submerged or emerging vegetation planted in soil, are used. Shallow water of depth 20-40cm flows through the system (Kadlec and Wallace, 2009, Vymazal, 2010). Water in a FWS CW is exposed as can be seen in Figure 2.1

Figure 2.1 Surface flow constructed wetland (https://permaground.wikispaces.com)

To ensure an even distribution of flow in the system, suitable structures are provided at the inlet and outlet. Anaerobic conditions usually prevail over the deeper waters and the substrate, while aerobic conditions occur at the surface of the water. Various types of wastewater have been treated with this type of CW, including municipal wastewater, mine drainage, urban rainwater, combined sewer overflows, runoff of agricultural areas, livestock and poultry wastewater and landfills (USEPA, 1999) and they are very effective for organic matter and nutrient removal especially over a longer retention time.

Some of the disadvantages of free water surface wetland include large surface area requirements; low winter temperatures in cold climate reduce the efficiency of removing harmful substances; due to the fact that anaerobic conditions prevail, the possibilities for some biological processes are very limited; mosquitoes and other insect vectors can pose a serious challenge (US EPA, 1999). There are cases where FWS CWs are not a suitable treatment option, such as in individual homes, parks, playgrounds or similar public facilities, because of the exposed water surface, which are accessible to humans and can be a breeding habitat for insect vectors. A gravel bed subsurface flow (SSF) CW can be a better choice for these applications.

The flow in a SSF CW can be in the horizontal or vertical directions. In a horizontal flow CW, water flows in a horizontal direction from the inlet to the outlet of the system, through the substrate material, encountering zones of different oxidation levels (Vymazal, 2008). Depth of water in SSF CWs is usually kept at or slightly below the level of the substrate material, as shown in figure 2.2. The porous medium supports the emerging aquatic vegetation. The substrate depth usually varies from 0.5-0.8m and while gravel of size 10-20mm are mostly used. They are called "Reedbeds" in Europe because the most commonly used macrophyte is common reed (*Phragmites spp*). For vertical flow subsurface systems, wastewater is dosed on the substrate surface at intervals, allowed to gradually drains through the substrate medium after which it is collected at the base. In between loading, the void spaces in the substrate are refilled with air thereby increasing oxygen transfer in the system.

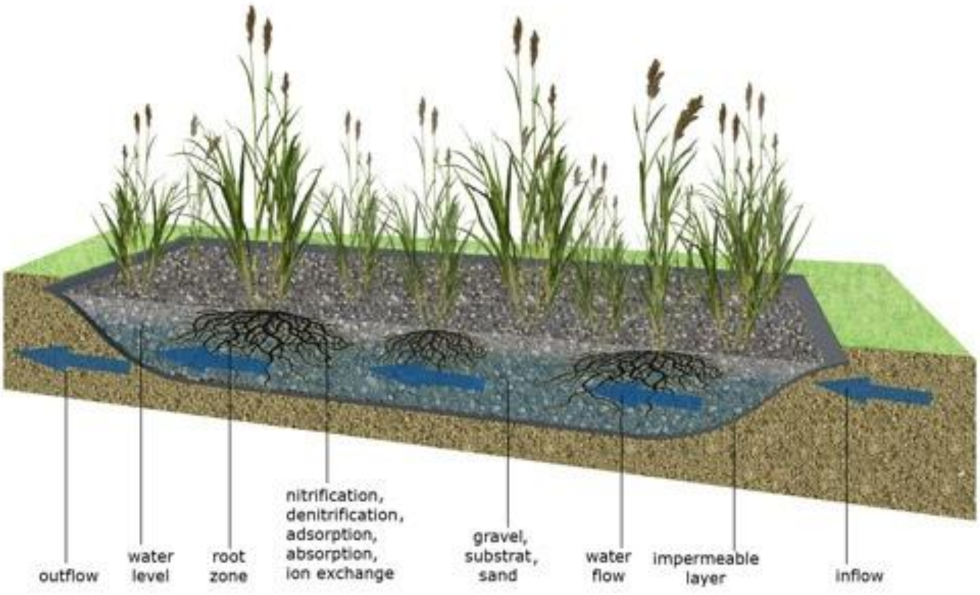


Figure 2.2 Subsurface flow constructed wetland (Vymazal, 2008).

SSF CWs are more appropriate for flows that are relatively uniform and effluents of low solids concentrations, because substrates can easily become clogged. The key advantages of SSF CWs include suitable for extreme cold conditions, low pest and odor problems and higher assimilation potential per unit of land area. Because the water surface is not visible, problems with public access are minimal. SSF CWs are more expensive to build compared to FWS CWs, as such and limit their use to small flows. The maintenance and repair costs are generally also higher. Clogging and unintended surface flow are also among the challenges of using SSF CWs.

In order to achieve higher pollutant removal efficiency, in particular with regard to the removal of nutrients, different types of CWs are combined and they are called "hybrid constructed wetlands (HCWs)". There are many configurations of hybrid systems, such as subsurface vertical flow followed by subsurface horizontal flow in series; surface flow followed by subsurface flow etc. The advantages of various types of CWs are combined to improve the wastewater treatment process and achieve efficiencies that may not be possible using single systems in isolation.

For instance, removal of nitrogen species (nitrification / denitrification) requires an aerobic / anaerobic state that can be provided by a combination of vertical flow wetland (aerobic state) and horizontally flowing wetland (anaerobic state) (Vymazal, 2005). Vymazal and Kropfelova, (2011) implemented a three-stage hybrid CW consisting of saturated vertical flow, free-drained vertical flow and horizontal subsurface flow wetlands, as shown in Figure 2.3

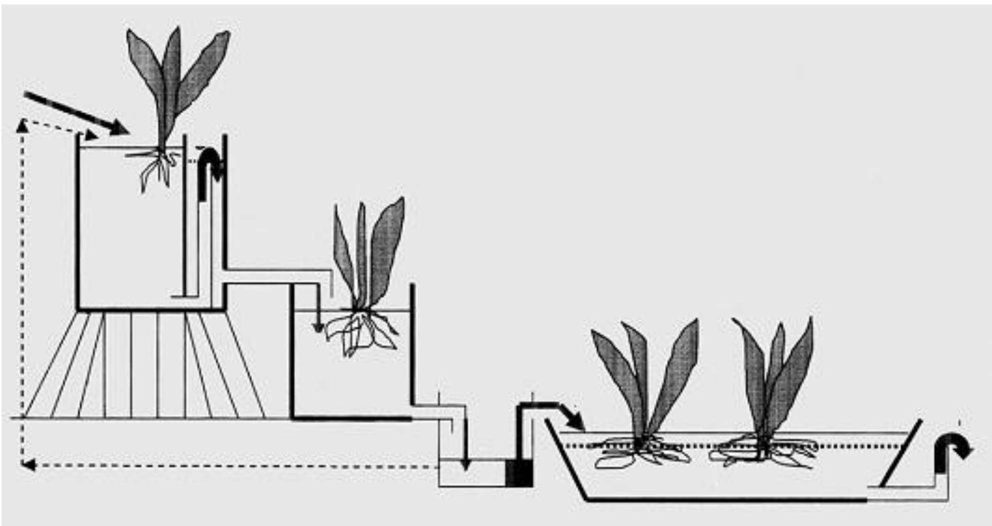


Figure 2.3 Three-stage hybrid constructed wetland (Vymazal and Kropfelova, 2011)

They stated that the system was very effective in reducing organics, suspended solids and nitrogen from municipal wastewater, with removal efficiencies of 94.5% and 84.4% for BOD and COD respectively; 78.3% for NH_4-N and 65.4% for Phosphorus. Another variant of the HCWs, the integrated vertical flow CW, which integrated down-flow and up-flow beds, was implemented by Chang *et al.*, (2012) and is presented in Figure 2.4.

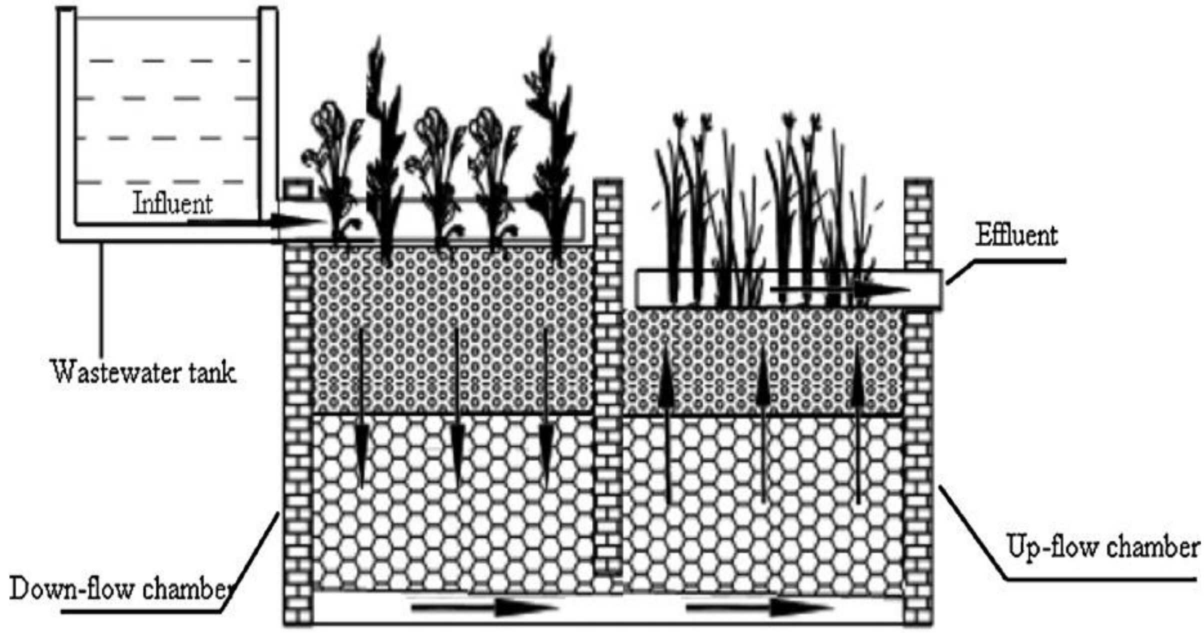


Figure 2.4 Integrated vertical flow CW (Chang *et al.*, 2012)

2.3.2 Constructed Wetland Components

To gain insight into the bio-remediation potentials and processes that occur in CWs, identification of the most important parts of the system is necessary. Four important components of most systems include the vegetation, substrate, water and micro-organisms. These components are discussed below.

2.3.2.1 Wetland Vegetation

Vegetation is an important part of a wetland system (Lee and Scholz, 2007, Kadlec and Wallace, 2008). Macrophytes play an important role in terms of contaminants reduction and general wastewater treatment. The roots, stems and leaves that extend through the water column provide enough attachment sites for microorganisms, and most of the treatment processes occur in the biofilms formed at these sites (Brix, 1997). Plants have been found to transport oxygen through their hollow tissues and leak oxygen into the wetland from their roots, creating an aerobic area in an otherwise anaerobic substrate, where aerobic degradation and nitrification can take place (Sim, 2003). These aerobic microbial populations are capable of modifying trace organics, nutrients and metal ions. However, the rate of oxygen mass transfer from plant roots is considered insufficient for the consumptive needs of aerobic heterotrophs that consume available organic substances (Vymazal, 2005).

Dead and decomposing plants are a well-known source of carbon for microbial population and different organic compounds are known to be released by macrophytes, which denitrifying microorganisms consume as food source (Brix, 1997). Wetland plants offer good conditions for physical filtration because they slow the water flow through the system, allowing settlement of solids and a higher contact time between wastewater and biofilms (Sim, 2003). Macrophyte leaves and stalks provide shades that limit the penetration of sunlight. This shadow effect tends to control algae growth by limiting light penetration to the water surface where they grow. Also the shadow effect of the plants has been found to lower the water temperature. Wetland plants also make wastewater treatment systems aesthetically appealing.

Selection of suitable plant species is important because the plants in wetland systems form the basis for the life of animals, as well as carrying out important hydrological and water-purifying functions. The selected plant species must be able to withstand the climatic conditions in which the CW is to be used, if annual planting is to be avoided. The most common emergent macrophytes are the *Phragmites spp*., *Typha spp*., *Juncus spp*. and *Scirpus spp*. (Lee and Scholz, 2007; Kadlec and Knight, 1996), while *Eichhornia spp*. and *Lemna spp*. are amongst the most common floating macrophytes (Sim, 2003).

Emergent macrophytes are herbaceous vascular plants with vast root and rhizome structure. They are rooted in the ground and appear and stand directly above the water surface from a depth of up to 1.5 m, although the flood tolerances vary by species. Emergent macrophytes can be propagated by seeds or by planting the rhizomes, and requires between two months to five years maturing, depending on the mode of propagation. According to USEPA (1999), *Phragmites* are the preferred emergent plants for subsurface systems in Europe due to the fact that they are a fast-growing hardy plant and are not a food source for animals or birds, but in the US they are not allowed because they are aggressive and there are concerns about their invasion of natural wetlands.

The treatment performance of different aquatic plants has been evaluated in many studies, with different conclusions. Dipu *et al*., (2010) compared the efficiency of Cattail (*Typha spp*.), Water hyacinth (*Eichhornia crassipes spp*.), *Salvinia spp*. and *Pistia spp*. in the treatment of effluents from a dairy factory and found the *Typha*-based treatment system most efficient in removing contaminants from the effluent.

Baskar *et al*., (2014) compared the organics and nutrients reduction efficiencies of *Phragmites australis* and *Typha latifolia* under different hydraulic residence times and found that *Phragmites australis* was better at removing organic substances and *Typha latifolia* was more efficient at removing nutrients. The higher organic removal capacity of *Phragmites australis* was attributed to the broad root zone and the enormous biofilm surface.

Burke (2011) assessed the carbon, nitrogen and phosphorus storage of *Scirpus acutus* and *Typha latifolia* with the intention of recommending one as a more effective choice for wastewater treatment based on the ability to store nitrogen and carbon and ability to retain nutrients during senescent periods, and discovered that *Scirpus acutus* was a more suitable species for wastewater treatment because it showed greater storage capacity and contributed less degradation by-products from tissues above ground than *Typha latifolia*. *Scirpus acutus* preserved more biomass and nutrients in tissues below the surface and decayed 30% slower.

Kumari and Tripathi (2015) reported that *Phragmites australis* and *Typha latifolia* significantly performed better when grown in combination than in monoculture for heavy metal (Cr, Fe, and Zn) removal.

2.3.2.2 Wetland Substrate

Substrates are an important component of CWs that affect construction costs, purification capacity and system maintenance costs. That is why substrate optimization is an important part of CWs research. Sand, gravel, stone and organic materials are the main media materials used in CWs. Substrate in wetlands provides physical support for plants, attachment sites for microbial populations and aid contaminant removal through a combination of processes such as sedimentation and filtration (Wang and Zhang, 2012). Gravel and sand are the most common substrate in the literature, because they provide the enabling environment for the removal processes that occur through biological and chemical means and also enhance the removal of solids and other polluting substances. Studies have investigated the purification capacity of these conventional substrates. Priya *et al*., (2013) investigated the nutrient removal efficiency of gravel and sand and found that sand provided a better removal of nutrients from wastewater than gravel, although the removal of TKN was better with gravel. Both sand and gravel were unable to remove NO_3-N from the system. They concluded that sand was more efficient than gravel.

The efficacy of non-conventional substrate material for use in CWs has been also investigated severally. Li *et al*., (2011) evaluated Calcium silicate, Vermiculite and Ceramsite for the removal of nutrients in CW systems and concluded that Calcium silicate showed better phosphorus reduction capacity (97%) compared to other substrates, while Vermiculite performend better for Ammonia Nitrogen reduction (65.91%), although they attributed this to the action of microorganisms. Ding *et al*., (2011) evaluated the treatment performance of fine sand, gravel, coal dust, slag and sewage sludge and concluded that reduction capacities of the substrates did not differ significantly.

Several studies on organic materials utilization as CWS substrate have been carried out. Wang *et al*., (2013) examined the pontentials of Oyster shell for the removal of phosphorus in swine wastewater and concluded that with the phosphorus absorption level observed in the wetlands, Oyster shell can serve as a CW media material for phosphorus removal. Wang and Zhang (2012) assessed the feasibility of using bamboo splint and palm silk as substrates for CWs and found contaminant reduction rates of 73.97%, 61.42%, 28.98% for COD, TP, TN respectively, using the bamboo splint substrate and 78.37%, 69.42%, 24.4% for the palm silk substrate, respectively, while control substrate had a removal rate of 66.61%, 58.71%, 22.23%, respectively.

The use of agricultural by-products as substrates in CWs has been evaluated. The motivation for the use of agricultural by-product is the fact that the cheapest substrates would be the unwanted substrates. Tee *et al*., (2009) evaluated gravel and rice husk based media for phenol and nitrogen reduction. The systems were planted with *Typha latifolia*. They discovered that the rice husks based wetland performed better than the gravel based wetland, which they attributed to the increased rhizomes in the rice husk based CW, which lead to an increase in the dissolved oxygen concentration thus creating increase aerobic conditions.

Cameron and Schipper (2010) reported that corn cobs were an excellent carbon substrate and produced significantly more Nitrate removal rates than wood substrates. Jong and Tang (2015) incorporated PKS as part of the substrate in a vertical flow CW for septage treatment in Malaysia and compared its organics and nitrogen removal efficiency to that of sand. They stated that the organic removal efficiency of the PKS CWs was satisfactory (> 90%) and that Nitrogen removal efficiency of above 91% obtained for the PKS bed was similar to the value of 95% obtained for sand. They concluded that incorporating PKS can help improve the effectiveness of Nitrogen removal in CWs.

2.3.2.3 Wetland Microorganisms

The wetland component that contributes significantly to the degradation of pollutants is the wetland microorganisms, which include bacteria, fungi, algae and protozoa. Wetlands provide an ideal environment for microbial populations because of the high content of nutrients and the water supply. The microbial biomass is an important source of organic carbon and many nutrients. These microorganisms consume contaminants as energy source for their survival. Microbial activities are either aerobic or anaerobic. However, a good number of bacterial species refered to as facultative anaerobes have the capacity to function under both conditions as environmental conditions fluctuate. Toxic pollutants like pesticides have a negative influence on the microbial community of a CW.

2.3.2.4 Water

Successful operation of a CW depends on the creation and maintenance of correct water depths and flows (Kadlec and Wallace, 2009). Wetland hydrology, which describes the movement of water into and out of a CW system, is considered the most important factor for the maintenance of wetland structure and function, the determination of plant species composition and the effectiveness of the treatment in a wetland project (USEPA, 1999). USDA *et al*., (1995b) stated that hydrology is a very essential performance factor in CWs because all CW functions are linked to water movement, and thus is considered the single most important factor that influences the success or failure of CWs.

The design of a CW is influenced by the amount of water and its movement through the system. Apart from serving as a means for transporting contaminants such as solids, water is also a basic requirement needed for biochemical reactions. The hydrology of CWs is strongly affected by the interaction with the climate and the weather (USEPA, 1999). Sources of water inflow into a CW includes, wastewater inflow, rainfall, overland flow and groundwater discharge, while outflow sources include, wastewater outflow, evapotranspiration (ET) and groundwater recharge. The Hydraulic residence time of a CW is influenced by precipitation, infiltration, evapotranspiration, infow and outflow rates and depth of water, and can also affect system performance in terms of removal rates by increasing the concentration of pollutants or by diluting the effluent. The total water input and output through a CW, is shown in figure 2.5.

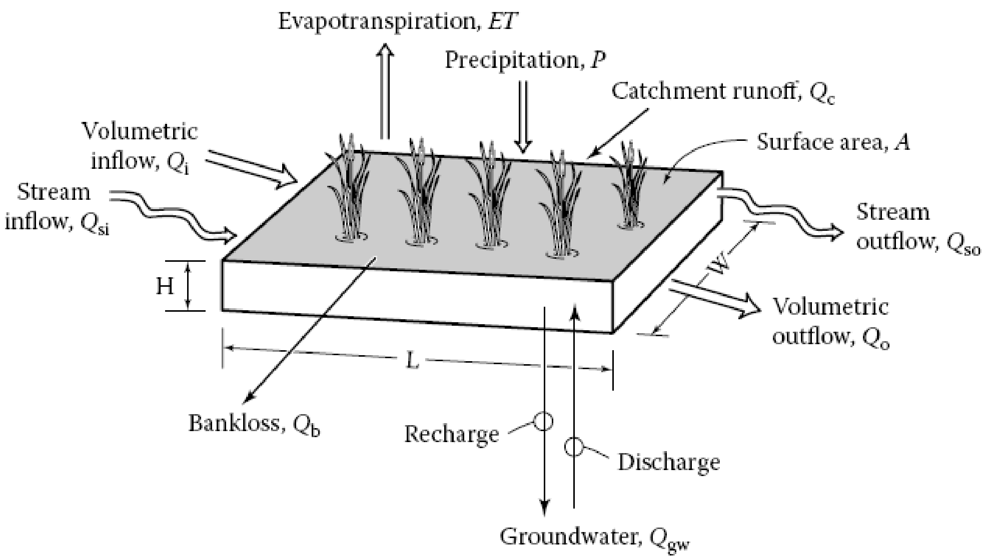


Figure 2.5 CWs inflow and outflows (Kadlec and Wallace, 2009).

The wetland water budget is given as (Kadlec and Wallace, 2009):

$$\frac{dV}{dt} = Q_i - Q_c + Q_{sm} - Q_o - Q_b - Q_{gw} + (P - ET)A_w \qquad 2.1$$

where V, t, Q_i, Q_c, Q_{sm}, Q_o, Q_b, Q_{gw}, P, ET and A_w are volume in m^3, time in days, inflow in m^3/d; catchment runoff in m^3/d, snow melt m^3/d, outflow in m^3/d, berm loss in m^3/d, groundwater infiltration in m^3/d, precipitation in m/d, evapotranspiration in m/d and surface area in m^2 respectively.

If a CW is properly compacted and lined with a water proof membrane, then Q_{gw} and Q_b can be avoided. Also for a tropical environment, Q_{sm} can be removed and Q_c can also be negelected. Therefore, Equation 2.1 reduces to (Kadlec and Wallace, 2009):

$$\frac{dV}{dt} = Q_i - Q_o + \left(P - ET\right)A_w \qquad 2.2$$

Hydrology of a CW is of crucial importance in the bio-remediation processes, because not only does the transport of contaminants to biochemical remediation sites depend on it, but it also affects the lenght of time water spends in the system (Kadlec & Wallace, 2009). Increase in retention time increases the likelyhood of greater bioremediation processes such as sedimentation and retention of nutrients. That is why the water budget of wetland is an important consideration in the design of CWs. All internal and external water sources must be included in the design of the system, including rainfall and ET. So precipitation and ET data must be included in the design (OhioEPA, 2007). The precipitation and evaporation data are measured on site or collected at meteorological stations. For CWs 0.8 times class A evaporation is normally used (Tousignant et al., 1999). Two to ten years of extreme dry and wet conditions are common in the literature for calculating water budgets (OhioEPA, 2007, Tousignant et al., 1999).

2.3.3 Pollutant Remediation Processes in Constructed Wetlands

As wastewater flows through a CW, various complex natural processes take place either concurrently or sequentially, whereby the contaminants are transformed and removed (USEPA, 1999), as shown in Figure 2.6. Filtration and sedimentation are amongst the key physical processes in CWs. Resistance to water movements in wetlands caused by plant roots and stems, reduce the velocity of flow thereby increasing sedimentation of suspended solids. Thus substrate materials act as filters (DeBusk, 1999a). Another very important mechanism for removing contaminants in CWs is the biological processes. Plant uptake and microbial metabolism are the most important biological processes for the removal of contaminants (DeBusk, 1999a). Plants absorb contaminants such as Nitrate, Ammonium and Phosphate as essential nutrients and can also accumulate toxic metals.

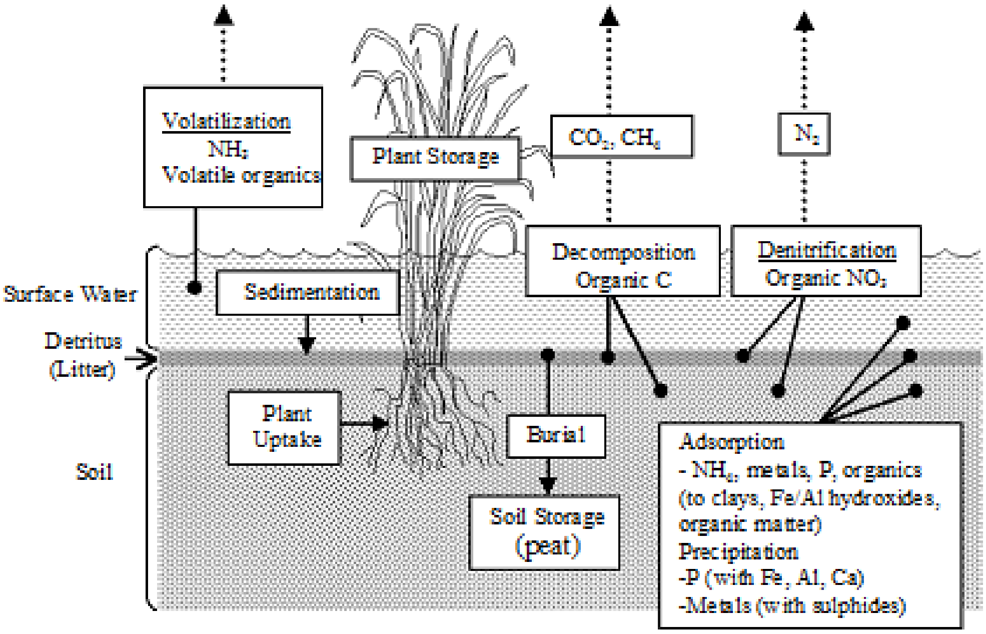


Figure 2.6 Major physical, biological and chemical processes controlling contaminant removal in constructed wetlands (DeBusk, 1999a).

However, these nutrients and metals are returned to the system through the decomposition of dead plant litter. Organic pollutants in CWs are mainly removed by microbial processes that occur in the biofilms formed in the wetland bed. The microorganisms, particularly facultative anaerobes, consume these organics to produce energy. The carbon in the wastewater is convereted to carbon dioxide when aerobic conditions prevail and methane under anaerobi conditions. Microorganisms also play an important role in the removal of inorganic nitrogen (DeBusk, 1999a).

Chemical mechanisms for removing contaminants include sorption, photo-oxidation and volatilization. Adsorption, which involves the transfer of charges in the soil and precipitation, which has to do with conversion of metals from soluble to insoluble form, are the key sorption processes that occur in CWs. Photo-oxidation involves the breakdown of compounds using solar energy. Volatilizaton on the other hand involves the breakdown and conversion of compounds into gaseous forms. All these processes form the basis for the removal of pollutants in wetlands. These mechanisms for removing major pollutants are discussed in the below paragraphs.

2.3.3.1 Nitrogen Removal Mechanisms

Nitrogen compounds are among the most important concerns in wastewater due to their role in eutrophication. Ammonium, nitrate, nitrite and organic nitrogen (proteins, peptides, nucleic acids and urea) are the major forms of nitrogen (USEPA, 1999). The major nitrogen removal mechanisms including ammonia volatilization, nitrification, denitrification, plant uptake etc are presented in Figure 2.7. The biological processes of nitrification and denitrification are the most important mechanisms (Vymazal, 2007).

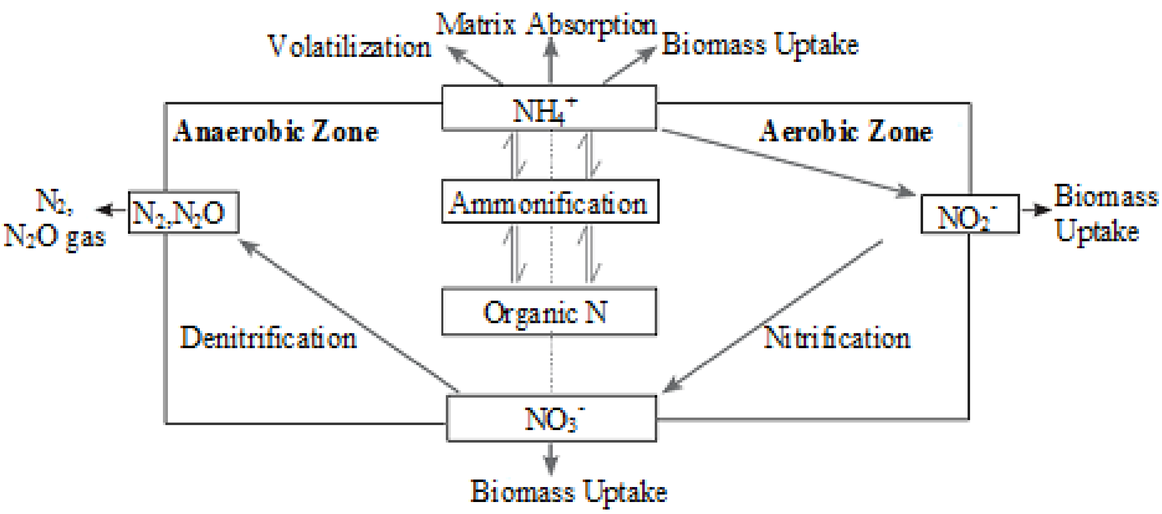


Figure 2.7 Nitrogen transformations in a constructed wetland (UN-Habitat, 2008)

A. Nitrification/Denitrification

Ammonification, which is the process of organic nitrogen conversion to NH_4^+ by hetrotrophs, is first step in the nitrogen removal process in a CW. It occurs both in aerobic and anaerobic environments, but it has been reported that it is much faster under the former condition (Vymazal, 2007), and depends of parameter such as temperature, pH and nutrients, with optimum range of 40-60^{o}C for temperature and 6.5-8.5 for pH (Vymazal, 2007). Ammonium is subsequently converted to gaseous ammonia or nitrified in the presence of oxygen (USEPA, 1999). After the conversion of organic nitrogen to ammonium, nitrification takes place in two steps. First, nitrogen fixing bacteria convert nitrogen into nitrite in the presence of oxygen, as given in equation 2.3 (Vymazal, 2007):

$$NH_4^+ + O_2 \rightarrow NO_2^- + 2H^+ + H_2 \qquad 2.3$$

Many bacteria have the capacity for such nitrogen transformations in the soil such as *Nitrosospira*, *Nitrosovibrio*, *Nitrosolobus* etc. The next step in the nitrogen removal process is the conversion of nitrite to nitrate given as (Vymazal, 2007):

$$2NO_2^- + O_2 \rightarrow 2NO_3^- \qquad 2.4$$

This process is carried out by the *Nitrobacter* bacteria, using nitrite as energy source. Parameters that influence the nitrification process are temperature, pH, alkalinity and oxygen content. For the process to proceed well, the alkalinity of the water and the concentration of dissolved oxygen must be optimal, and has been reported to consume about 4.3mg and 7.14mg of dissolved oxygen and alkalinity for every mg of ammonia oxidized (USEPA, 1999). Also temperature should be in the range of 30 to 40^{o}C and pH in the range of 6.6 to 8.8. Next is the reduction of nitrates to organic nitrogen in a anaerobic process refered to as denitrification (equation 2.5). *A pseudomonas spp.* bacterium catalyzes the process. Nitrogen is eventually removed from the CW by the release of nitrous oxide into the atmosphere (Vymazal, 2007).

$NO_3^- + C\ (organic) \rightarrow N_2 + CO_2 + H_2O$ 2.5

Denitrification mainly occurs at sites of low oxygen content in CWs, especially in biofilms formed in the substrates below the developed roots of macrophytes (USEPA, 1999). Decomposing wetland plants and plant roots, especially at the beginning of senescence, provide a source of biodegradable organic carbon for denitrification. For every gram of nitrate removed, about 2.86g and 3g of dissolved oxygen and alkalinity respectively are used (USEPA, 1999). The denitrification process is dependent on several parameters such as temperature, carbon source, dissolved oxygen, presence of denitrifying bacteria etc (Vymazal, 2007).

B. Ammonia volatilization

Aqueous ammonia volatilizes to ammonia gas (NH3) at the air / water interface and is released into the atmosphere. Ammonia volatilization in wetlands is of limited importance, except in cases where ammonia nitrogen concentrations are higher than 20 mg/l. Kadlec and Knight (1996) revealed that volatilization is normally unimportant at a pH below 8. In general, in CWs treating animal wastewater, BOD is usually high and dissolved oxygen levels normally low, and as such, oxygen should not be high enough to nitrify ammonia. If nitrification / denitrification in wetlands is low, ammonia evaporation can explain the reduction of nitrogen (Poach et al., 2003).

C. Plant intake (assimilation)

Plant uptake has also be recognized as a nitrogen removal mechanism in CWs. Plants absorb and store nitrogen in the organic form. DeBusk, (1999a) stated that macrophyte growth rate and the level of contaminants already absorbed by the macrophytes are some of the important factors that affect the removal capacity of plants. Higher nitrogen removal is usually observed during macrophyte growing periods. However, the assimilated nitrate and ammoniacal nitrogen can also be recycled in the CW through biodegradation of plants. CW vegetation is thus only regarded as a temporary nitrogen sink, unless the biomass is harvested (Brix 1997). The absorption capacity of emergent plant species in CWs varies from 200 to 2500 $Kg.ha^{-1}$ $year^{-1}$ (Brix 1997).

D. Matrix Adsorption

Detritus and inorganic materials or substrates cation exchange can cause ionized ammonia adsorption from a solution. Reduction of the level or concentration of ammonia in the water by nitrification to regain equilibrum can cause loosely bound adsorbed ammonia to be desorbed. However, increase in the concentration of ammonia in the water can cause an increase in the adsorbed ammonia. The adsorbed ammonium can be oxidized when the aqueous substrate is exposed to oxygen (Kadlec and Knight, 1996). Factors like clay content, type and quantity of organic material, detention time etc matrix adsorption in a CW.

2.3.3.2 Phosphorus Removal Mechanisms

Phosphorus is considered to be the limiting nutrient associated with eutrophication. They usually exist as phosphates either in solution or in particulate form, and the main classifications include orthophosphates, condensed and organically bound ones (USEPA, 1999). Phosphorus removal is usually low in wetlands compared to nitrogen removal because there are no direct metabolic pathways for phosphorus removal. However, several physical, chemical and biological processes are usually involved in the transformation of phosphorus (DeBusk, 1999b). Figure 2.8 illustrates the phosphorus cycle and its fate in wetlands.

Plant uptake, attachment in developed biofilms and sedimentation are the major physical removal routes. Most of the phosphorus removal via plants take happens when the plants are young and growing. Storage in the roots and rhizomes are higher than in the stems and leaves and is also influenced by the macrophyte type in the CW. Phosphorus is usually recycled in the system with absorption and release happening at different time in the system. Decomposition of the above ground biomass releases phosphorus into the water, while decay of the roots and rhizomes releases phosphorus into the substrate.

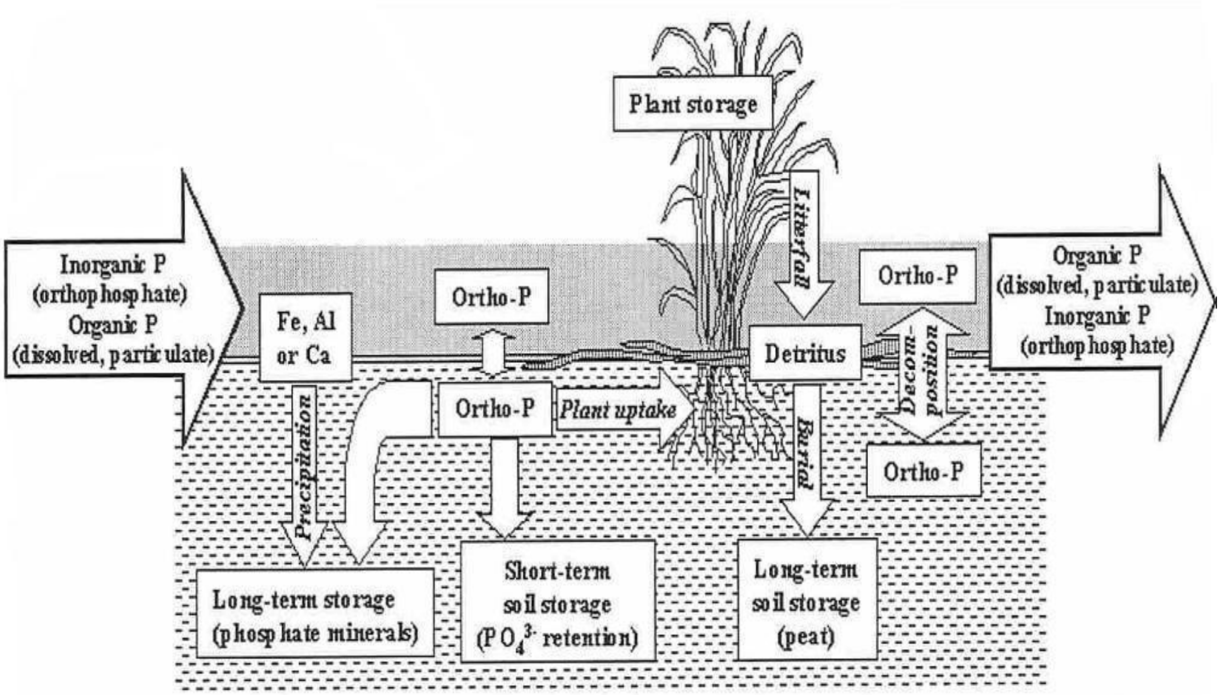


Figure 2.8 Fate of phosphorus in wetlands (DeBusk, 1999b).

Chemical methods are soil adsorption / desorption and precipitation. The most important route for soluble phosphates in wetlands is through its exchange by absorption /desorption in the pore water spaces (USEPA, 1999). Garcia et al., (2010), however, stated that adsorption of phosphate by the substrate material is mainly a function of the texture, particle size distribution and the Fe content. Substrate adsorption capacity is also influenced by the clay content of the substrate as well as its mineral content (Vymazal, 2007). Kadlec and Wallace (2009) opined that adsorption and desorption of phosphates are also dependent on the pH, redox potential and the mineral constituents in the sediment.

Another important process is chemical precipitation. New compounds are formed by chemical presipitation, such as Iron and aluminium phosphate formed from iron and aluminum oxide precipitate. The formed compounds are very stable, allowing for phosphorus to be stored over a long period (DeBusk, 1999a). However, anaerobic conditions can lead to the disolution of some of these compound, leading to the release of orthophosphate into the water however by hydrolysis (Garcia et al., 2010, USEPA, 1999). Chemical removal of phosphorus is usually very high in the early stages of treatment (possibly longer than a year), but this high efficiency of treatment disappears normally after this period (USEPA, 1999)

According to Vymazal, (2007), SSF CWs are more suitable for adsorption and precipitation of phosphorus, compared to FWS systems because wastewater comes into contact with filtration substrate and the constant water in the SSF system unlike the load and drain mode of FWS systems, which leads to bed oxygenation, ensures there is little redox potentials fluctuations that adsorption and desorption of phosphorus. Many common CWs substrate materials like gravel, usually offer very low capacity for sorption and precipitation.

Microbial breakdown of phosphates is a very quick biological process because of the very rapid multiplication in the population of microbes; however they do not have the capacity to store very large quantities of phosphorus (Vymazal, 2007). Also this process only occurs on a temporal basis as the phosphorus stored by these bacteria, fungi and algae communities is quickly released back into the water following their death and decomposition (Vymazal, 2007).

2.3.3.3 Total Suspended Solid Removal Mechanisms

Total suspended solids removal in CW is achieved by a number of processes such as sedimentation, filtration, adsorption and flocculation/precipitation. In free water surface wetlands, TSS is mainly removed by flocculation/sedimentation and filtration/interception as shown in Figure 2.9.

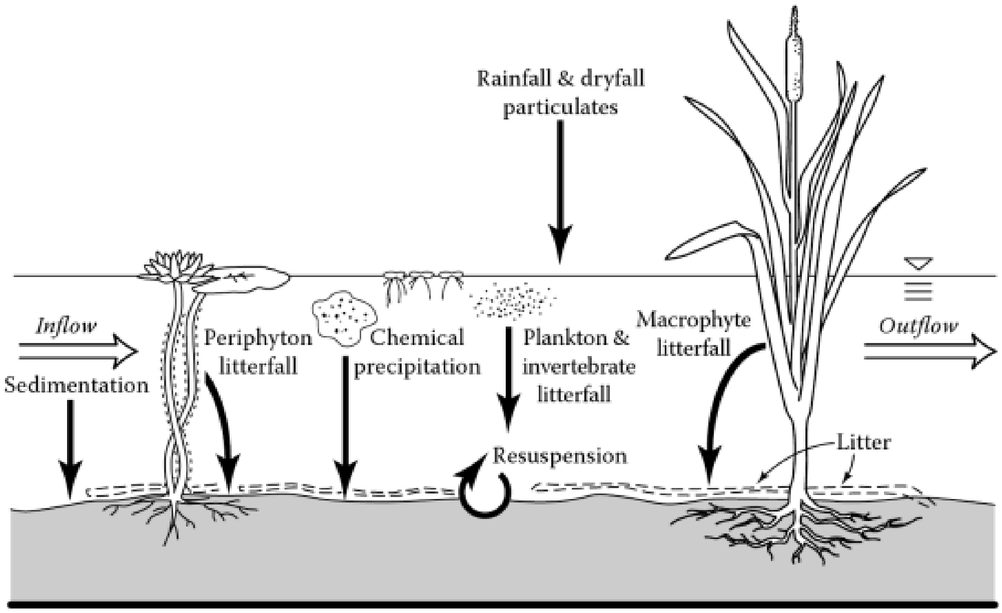


Figure 2.9 TSS removal and generation in wetlands (Kadlec and Wallace, 2008)

Various factors, including particle size, shape, specific gravity and macrophyte species, influence these removal processes. Interception and attachment to plant surfaces can also be seen as another important process in the removal of TSS. The periphon, which are biofilms formed on plants surfaces can absorb colloidal and soluble substances, which are then metabolized into gases or biomass (USEPA, 1999).

Flocculation and filtration in the granular bed are the main mechanisms of TSS removal in SSF CW. These mechanisms are relatively effective due to the low velocity of flow rate and large bed surface area. Gravity sedimentation, filtration and adsorption of solids on tha attached biofilms on gravel and root system is established in SSF CWs (USEPA, 1999). Clogging is a major problem when removing TSS because it reduces the hydraulic conductivity of the substrates. Different media sizes are mostly used in the influent and treatment zones to minimize their effects. Resuspension of settled solids can mainly take place due to disturbances caused by animals, high flow velocities, winds, bubbling of air in the system, and release of gases produced during nitrogen removal processes. (Kadlec and Knight, 1996; DeBusk, 1999a; USEPA, 1999).

2.3.3.4 Organic Matter Removal Mechanisms

Physical and biological processes are responsible for the reduction of organic materials in a CW. Wastewater contains particulate organic matter as well as dissolved and colloidal organic matter. This influent particulate organic materials can be entrapped in the biofilms or accumulates on the bottom if the CW. In addition, the organic matter that comes from dead plants can also collect at the bottom of the wetlands. Same mechanisms for TSS removal are also responsible for the separation organic matter in CWs (USEPA, 1999).

Adsorption and absorption are important mechanisms for the removal of soluble organic substances. These soluble materials are more likely adsorbed to biofilm on the plant surface and consumed by microorganisms, and the end product of this process depends on the oxygen state in the system. The extent of sorption and its speed depend on the properties of the organic substances and the solids. Volatilization can also explain the loss of certain organic substances. However, organic substances entering a wetland after the primary treatment do not contain significant quantities of volatile solids (USEPA, 1999).

Biological reactions such as oxidation/reduction, hydrolysis and photolysis controls organic content of CWs (USEPA, 1999). During the microbiological degradation, aerobic heterotrophic bacteria consume oxygen and break down the organic matter. Therefore, insufficient oxygen supply will significantly reduce the biological oxidation process. Anaerobic degradation of organic matter occurs in the absence of dissolved oxygen and yields methane as end product. It also leads to the production of gases and biomass, which are hydrolyzed to produce organic compounds that are more soluble in water. These compounds are oxidized to CO_2, various nitrogen and sulfur forms and water, but are converted to organic acids and alcohols in the absence of oxygen.

2.3.3.5 Metals and Pathogens Removal Mechanisms

Trace quantities of metals like copper, selenium and zinc that necessary for plant and animal growth and development, but are very dengerous when they exceed a certain concentration. Others such as cadmium, mercury and lead are outrightly harmful no matter the concentration. (USEPA, 1999). Mechanisms for metal removal in CWs are uptake by macrophytes, adsorption to substrates and chemical precipitation. Pathogens that are often found in untreated wastewater include helminths, protozoa, fungi, bacteria and viruses. Pathogenic removal in wetlands is a complex process because it depends on many processes such as natural death, prevailing temperatures, solar intensity, sedimentation etc. Some pathogens are removed when the perticulate materials they are attched to settles, while others are removed by predation as the CW plays host to quite a large variety of microbial communities, such as zooplankons which are pathogenic predators. Ultraviolet radiation from the sun that easily penetrate the shallow waters in CWs also accounts for some pathogen destruction. FWS wetlands on the surface of the water work better in this respect than SSF wetlands, because organisms are removed near the water surface by UV radiation (USEPA, 1999). UV radiation is a powerful means of killing bacteria in wetlands, but the fraction of incoming solar radiation in the UV range is small (Kadlec and Wallace, 2008). There are key indicator organisms like coliforms that the pathogen reduction efficiency of CWs are based on. This is because monitoring the very many pathogenic microorganisms in not possible. These indicators are easy to control and correlate with populations of pathogenic organisms.

2.3.4 Performance of Constructed wetlands

The treatment capacity of different types and configurations of CWs, in which different types of wastewater are treated, have been extensively studied, but different results are reported in the literature. In general, wetlands have been reported to perform very well in terms of BOD, COD and pathogen reduction, but have shown a limited capacity for nutrient reduction.

Adeniran *et al*., (2014) in their research on the performance and efficiency of a CW reported a 97% reduction for TDS, 99% reduction for turbidity, 89% reduction for manganese. While the values recorded for nitrate, sulphate, iron, BOD and *E. coli* were 92%, 42%, 97%, 82% and 99% respectively. There was a 138% increase in the dissolved oxygen content and an increase in the pH of the CW from 6.4 to 7.05.

Mairi *et al*., (2012) assessed the performance of SSF CW for the treatment of domestic wastewater and reported very high removal rates for total *E. coli* and faecal, with values as high as 93% for the planted cells and 75% for the unplanted cells. The planted cells significantly performed better than the unplanted cells in terms of BOD reduction, with a difference in removal efficiency of about 26%. Similarly, removal of nutrients was significantly higher in planted cells compared to unplanted cells.

Dhulap and Patil (2014) assessed the removal of pollutants from sewage with CWs and reported a maximum EC reduction of 34.61%, TSS reduction of 55.17%, TDS reduction of 56.18%, TS reduction of 55.48%, BOD reduction of 76.65% and COD reduction of 77.51%. The maximum NO_3, PO_4^3, SO_4 reduction rates were 74.62%, 57.81% and 51.06%, respectively.

Badejo *et al.,* (2012) reported a reduction of 82.0% and 85.0% for BOD, 72.0% and 73.0% for TDS, 78.0% and 81.0% for PO_4^3, 61.0% and 65.0% for NO_3 for *Vetiveria nigritana and Phragmites karka* respectively in their assessment of tertiary hospital wastewater treatment using constructed wetlands in Nigeria.

Nzabuheraheza *et al.,* (2012) in their evaluation of CW performance in Tanzania found that subsurface horizontal flow systems with *Cyperus spp.* and *Phragmites spp.* had reduction efficiencies of 72%, 80%, 81% and 78% for TDS, TSS, COD and BOD respectively, compared to a while a second monoculture wetland with *Cyperus spp.* had reduction efficiencies of 71%, 79%, 73% and 75% for TDS, TSS, COD and BOD respectively.

Oginni and Isiorho (2014) in their evaluation of CW treatment of wastewater in a residential tertiary institution in Nigeria observed very low removal rates (8% and 11%) for TDS and conductivity. While pH remained constant at 6.8. However, high removal rates were observed for microbiological contaminants which include 85% reduction for coliform, 79% for *staphylococcus*, 52% for *salmonella*, 79% for *salmonella* and *shigella*, 66% for total viable count and 83% for fungi.

Chang *et al.,* (2012) in their study of the treatment performance of integrated vertical-flow CW plots for domestic wastewater reported average reduction efficiencies of 59% to 62% for COD, 12% to 15% for TN, and 51% to 52% for TP.

Kadlec and Knight (1996) evaluated the average performance of 70 North American surface flow CWs treating domestic or agricultural effluent. They reported average values of 74% for BOD, 70% for TSS, 54% for NH_4-N, 61% for NO_3 and 37% for Orthophosphate. The removal rates were observed to be higher for BOD and solids, than for nitrogen and phosphorus.

Gutierrez *et al.,* (2004) monitored pollutant reduction in full-scale horizontal subsurface flow CW treating slaughterhouse effluent in Mexico and concluded that the system achieved satisfactory pollutant removals but the effluent could not meet the Mexican environmental regulations for fecal coliform, BOD and TSS.

Varying removal efficiencies have been reported for various metals. Khan *et al.,* (2009) evaluated the use of CWs for heavy metals removal in Pakistan and observed a removal efficiency of 50%, 91.9%, 74.1%, 40.9%, 89% and 48% for Lead, Cadmium, Iron, Nickel Chromium and Copper respectively.

Chen *et al.,* (2009) evaluated the heavy metals removal efficiencies of CWs with coke and gravel, and reported that two wetlands had good lead removal efficiencies, with levels in the range of 95-99%. 54-91% and 69-99% reduction efficiencies were obatined for Zinc and Copper respectively, which suggested that the efficiency for Lead was higher than the efficiencies for Zinc and Copper.

Sahu (2014) assessed the reduction of heavy metals from wastewater using CW and reported a reduction of 43% for Hg, 46% for Fe, 49% Ni and 54% for Cr. Maine *et al.,* (2006) evaluated metal uptake using a combination of different plant species namely water hyacinth (*Eichhornia crassipes*) cattail (*Typha domingensis*), and elephant panicgrass (*Panicum elephantipes*) in a constructed wetland, and reported reduction efficiencies of 86% for Cr and 67% for Ni and 95% for Fe.

2.4 Design Considerations for Horizontal Subsurface Flow Constructed Wetland

CWs design is a critical phase of the implementation process, because it becomes very difficult to correct any design mistakes after the system has been built (Rani et al., 2011). There are several published guidelines for HSSF CW design and construction, including Reed *et al.* (1995a); OhioEPA (2007); Kadlec and Knight (1996); USEPA (1999), but guidelines have not been set for humid tropical climates. The main design considerations for HSSF CW are quality and quantity of the wastewater, surface area, bed depth, length / width ratio (USDA *et al*., 1995a, USEPA, 1999). Other equally important parameters that must be taken into account are the bed slope, the media type, the lining, and the inlet and outlet distribution systems.

- **Wastewater Quality and Quantity**

Wastewater evaluation is the first step in the design of an HSSF CW. The influent concentrations of the various relevant parameters must be known before the start of the design process, because this can inform the type of treatment systems that are needed (Tousignant *et al*., 1999). A very critical CW design consideration is also the accurate determination of effluent volumes. Daily, weekly and monthly averages from all know input sources and for the different seasons of the year must be estimated. This is very important for water budget analysis (Tousignant *et al*., 1999).

- **Surface Area**

The successful use of CW technology to for effluent remediation is dependent mainly on the correct sizing of the system and also proper operational specifications. Different approaches have been used to size treatment wetlands. For example, there have been cases where the design of CW was based on daily mass loading of pollutants. In this case the contaminant mass in kg/day, which is a product of the daily flows and contaminat concentration, is used to determine the size of CW, using loading rates in kg/ha/d specified in design guidelines (Tousignant *et al*., 1999).

Different models have been developed and used to design and predict the removal of contaminants in a CW. The state-of-the-art design method consists of equations of the first order, assuning plug flow of water in the CW (Rousseau *et al*., 2004). First order kinetics means that contaminat removal rate is directly proportional to the residual concentration, while plug flow conditions refers to decrease in the concentration of pollutant with length in the CW. Designing CWs with the first order plug flow kinetic design model only yields conservative estimates as it is a well established fact that the hydraulic regime in wetlands is inbetween plug flow and completely mixed (Rousseau *et al*., 2004).

Reed *et al*. (1995a) proposed that for contaminants such as BOD, NH_4 and NO_3 that their remedaition are purely through biological means, the first order plug flow kinetic model can be used to describe the treatment processes. But for other contaminants such as TSS and TP, they proposed the use of regression equations derived from the analysis of previous performance data. The design model based on Reed *et al*., (1995a) is shown below:

$$A = L{*}W = \frac{Qt}{yn} = \frac{QIn\left(\frac{C_i}{C_e}\right)}{K_v yn} = \frac{QIn(C_e - C_i)}{K_T yn} \qquad 2.6$$

Where A is area (m^2); C_e is outlet pollutant concentration (mg/l); C_i is inlet pollutant concentration (mg/l); K_v is volumetric rate constant (day^{-1}); L is length (m); W is width (m); d is depth (m); n is porosity (%); Q is flow rate (m^3/d); t retention time (d^{-1}).

In recent times, an adapted first-order kinetic model developed by Kadlec and Knight (1996) and has become very popular for sizing wetlands. The model commonly called the *k-C** model is a reversible first-order reaction equation for all pollutants, including organic substances, solids, nutrients and pathogens, and takes into account the fact that the concentration of contaminants in the wastewater cannot be reduced to zero as a result of the subsequent release of pollutants from the wetland into the treated water. The surface area of a CW based on the *k-C** model is given as (Kadlec and Knight, 1996):

$$A = \frac{Q}{K_A} In\left[\frac{C_i - C^*}{C_e - C^*}\right] = \frac{Q}{K_v dn} In\left[\frac{C_i - C^*}{C_e - C^*}\right] \qquad 2.7$$

Where A is CW area (m^2); Q is flow rate (m^3/day); C_e is target effluent concentration (mg/l); C_i is the influent concentration (mg/l); K_A is areal rate constant (m/day); K_v is the volumetric rate constant (day^{-1}); C^* is residual concentration (mg/l).

Another first-order model improvement was proposed by Shepherd *et al*., (2001). The time-dependent retardation model replaced the background concentration C^* with two other parameters K_o and b. The model assumes a decrease in removal rates over time, due to the fact that some biodegradable pollutants are quickly degradable, while pollutant of lesser biodegradable nature are left in the system and takes longer time to be degraded. This fluctuation of solution composition is represented by a time dependent first-order removal rate constant given as:

$$K_v = \frac{K_o}{b\tau + a} \qquad 2.8$$

Where K_v is rate constant (d^{-1}); K_o is initial rate constant (d^{-1}); b is retardation coefficient (d^{-1}) and τ is residence time (d).

Because of the fact that the retardation model allows a continuous decrease of contaminants with a longer treatment time instead of a constant residual value (C^*), it is considered suitable for wetland design (Rousseau *et al.*, 2004).

Studies have shown that the hydraulic regime in most wetlands as far from being ideal and that the first-order kinetic plug flow models discussed above do not adequately represent the removal mechanisms and flow behaviour of CWs. The tanks-in-series (*TIS*) model has been used to predict the fate of pollutants in CWs. The TIS model uses the parameter N, which represents the effective number of tanks. N is found by tracer studies. Another proposed approach for predicting pollutant removal and modeling wetlands is using the the *P-k-C** model. The model is probably the most recent kinetic method of system design and prediction of pollutants. It is recommended for modeling parameters consisting of different compounds, such as organics, solids or nutrients, and the treatment of wetland performance is well represented by the *P-k-C** model (Kadlec and Wallace, 2009). The CW area based on the *P-k-C** model is given as (Dotro *et al.*, 2017)

$$A = \frac{PQ_i}{K_A}\left(\left(\frac{C_i - C^*}{C_e - C^*}\right)^{\frac{1}{P}} - 1\right) = \frac{PQ_i}{K_v h}\left(\left(\frac{C_i - C^*}{C_e - C^*}\right)^{\frac{1}{P}} - 1\right) \qquad 2.9$$

Where *P* is tanks-in-series number.

The main disadvantage of this approach is the fact that many variables have to be assessed and most have not been extensively studied to generate design information and data under different situations. The existing design information were developed for cold climatic conditions, and thus cannot be used for design in tropical environmental conditions. Also the number of tanks in seires is dependent on the CW geometry, and as such it must be taken into consideration. (Dotro *et al*., 2017)

Many better refined models for CW design and performance evaluation have been recently proposed and also many successful mechanistic models have been developed to simulate the complex integrated processes that occur in such a system and also in view of the non-ideal hydraulics observed in many CWs. However, they have not been adopted by practitioners because the presence of a large number of empirical parameters often leads to practical application difficulties (Rousseau *et al*. 2004) and their verification has not yet led to improved performance compared to some more simple empirical models that require less calibration parameters. Rousseau *et al*., (2004) discussed the current CWs design approaches and concluded that the *k-C** model remains the best available method, despite the obvious shortcomings. Stein *et al*., (2006) also emphasized the use of first order kinetic models for sizing wetlands, which is also in line with entries from Son *et al*., (2010) who believed that data availability and applicability have continued to favour the utilization of very simple models for wetland design and performance prediction.

- **Bed Depth**

There is no consensus about the proper depth of the substrate for HSSF CWs. A study found a slightly better removal of the BOD with a greater media depth, when comparing 0.45 m with 0.3 m systems that were operated at the same areal load (USEPA, 1999). Garcia *et al*., (2004a) evaluated the performance of CWs with different depths and found that shallow, horizontal flow wetlands were more effective than deep ones. Basing system depth on the maximum root penetration of macropytes has been proposed, so that the flowing water can have sufficient contact with the root of the plant for an effective treatment. Typical average media depths in HSSF systems are in the range of 0.3 to 0.7 m, but a range of 0.4 to 0.6m have been generally recommended and the upper end of the range is considered the best (USEPA, 1999).

- **Aspect Ratio**

The length to width ratio is important as their dimensions are multiplied to obtain the surface area and it is considered critical in maintaining sufficient flow through the wetland. The longer and narrower a flow channel is, the closer the flow is to plug conditions and vice versa. It has been observed that very high length to width ratio increases the hydraulic retension time and can also cause overflow challenges, esspecially as a result of buildup of plant litter over time (Kadlec & Wallace, 2009). Length to width ratios of between 2:1 and 5:1 are common in the literature. However, the IWA (2000) technical report suggests that any aspect ratio can be applied with a good inlet distribution, as previous assumptions that wetlands with high aspect ratios would work more efficiently and closer to the plug flow have not been confirmed in tracer studies. When subsurface flow conditions are expected in the wetland bed, it is common practice to use Darcy's law, which describes the flow regime in a porous medium (USEPA, 1993). It determines the flow that can pass through the bed in underground conditions. Darcy's law is usually given as (USEPA, 1993):

$$Q = K_S A_C S \qquad 2.10$$

Where Q is the flow per unit time (m^3/d), K_S is the hydraulic conductivity of a unit area of the medium perpendicular to the flow direction ($m^3/m^2/d$), S is the hydraulic gradient of the water surface in the flow system ($\Delta h/\Delta L$, m/m). A_c is the total cross-sectional area, perpendicular to flow (m^2) and is obtained by multiplying the width and depth of the system (USEPA, 1993).

According to USDA *et al.,* (1995b) the width of the subsurface flow CW is determined after calculation of the system area and selection of an optimal aspect ratio. For example, if a ratio of 3:1 is selected, then the width is determined from the Equation 2.11:

$$A = 3W^2 \tag{2.11}$$

- **Bed Slope**

USEPA (1993) stated that bottom slopes of up to 8% were used for CWs with subsurface flow in Europe. There is no consensus on an optimal slope, but most studies recommend a slope of between 0.5 to 1% so as to avoid construction difficulties and for good drainage (Kadlec and Knight, 1996). The most important practical consideration is to have uniform slope at the bottom, from the inlet to the outlet. According to USDA *et al*., (1995b) "the bottom of the cell may be flat or slightly inclined from bottom to top, but the surface of the medium must be level regardless of the slope of the bottom".

- **Media Types**

Different media types are used for HSSF CWs, varying from medium gravel to coarse rock (USEPA, 1993). Particles of small sizes do not perform well as wetland media because of their very low hydraulic conductivity and they can also cause surface flow. On the other hand, if the particle sizes are too big, they may not provide enough surface area for attachment of microorganisms but although they have good high hydraulic conductivity (IWA, 2000). The commonly used substrate material in Europe are sand and gravel (IWA, 2000). Determination of porosity and conductivity are amongst the key preliminary investigations necessary prior to the design of subsurface flow wetlands. Different gradations of media must be used at different locations within the HSSF CWs, because they will offer different functions. As shown in Figure 2.10, substrates of size between 40 and 80mm have been recommended for use at the inlet distribution and outlet collection points to reduce the occurance of clogging, with maximum lengths of about 2m. Subrtates, while for the treatment zone substrates sizes in the range of 20 to 30mm have been recommended USEPA, (1999). HSSF CWs with media size between 5 mm and 20 mm are, however, common in the literature.

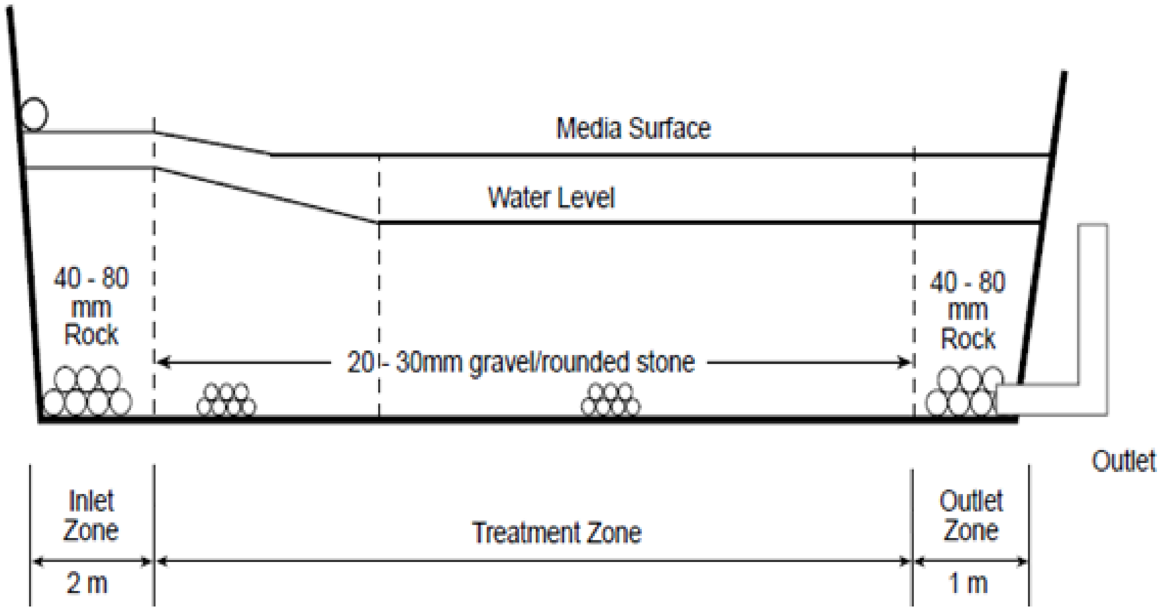


Figure 2.10 Zones in subsurface horizontal flow constructed wetland (USEPA, 1999)

- **Liners**

Subsurface flow CWs are usually sealed to ensure that water does not leak out of the system and contaminate the groundwater. Materials used for the liner include compacted clay, synthetic materials (polyethylene) and bentonite. Synthetic liners are not necessary if the native soil limits the water movement in a similar manner (i.e., if the permeability coefficient is less than or equal to 1 x 10^{-7} cm/sec). Indigenous soils can be compacted to prevent leakage provided the clay content is sufficient enough (≥ 15% clay) to give the desired permeability. Materials like portland cement and bentonite can be added to indigenous soils to improve the permeability and compacted to prevent leakages. The thickness of polyethylene normally used for coating wetlands ranges from 0.8 to 2.0 mm. Polyethylene liners are usually protected on both sides by geotextile or sand to prevent root penetration and damage from sharp objects.

- **Inlet and Outlet Structures**

Facilities for the control of water distribution and collection are crucial for the overall success or failure of a CW system. The inlet control structure must distribute the inflow evenly in the CW, while the outlet structure must uniformly collect wastewater effluent from the CW and also serve as a water depth regulator. In most cases, multiple inlets and outlets are needed to gaurantee adequate and uniform water distribution. Proper design of the inlet and outlet will minimize the existence of zones of poor water exchange know as dead zones, and also minimize prefrential flows, therby ensuring that the residence time of water in the system is significantly different from the theoretical residence time.

System configuration also determines the type of inlet and outlet structures to be used. Several factors, such as the amount of space available, the contours of the site, surface limitations that do not permit the placement of a system and other issues specific to the site, can influence the layout of an HSSF CW (OhioEPA, 2007). Configurations commonly found in the literature on CWs include: series cell flow; multiple cells in parallel flow; and series serpentine flow. According to OhioEPA (2007), the configuration of a CW must contain at least two wetland cells in series to maximize the amount of treatment that the system can offer. This minimum criterion is not only to offer more treatment, but also to allow flexibility in case of maintenance problems

2.5 Hydrodynamics of Subsurface Flow Wetlands

2.5.1 Hydraulics

The hydraulic loading rate (HLR) of a CW refers to the volume of wastewater loaded over a specified wetland area and time. It is defined as the mean flow rate divided by area. The hydraulic loading rate (m/d) is expressed as (Tousignant *et al.*, 1999):

$$HLR = \frac{Q_i}{A} \qquad 2.12$$

Where Q_i is the inflow (m^3/d), A is the wetland top surface area (m^2).

An increase in HLR increases the quantity of contaminants that flows through the wetland, giving a high mass removal rate per unit time because there is an increased availability of contaminants for microorganisms. However, this increase will lead to a decrease in the residence time of wastewater in the system, resulting to a lower relative removal of pollutants. If the increase in water flow through the system does not result to a significant increase mass loading to the system, the increase in flow can be ignored (Tousignant *et al*., 1999).

The nominal volume (V_n) is the volume of the CW multiplied by substrate porosity and is given as:

$$V_n = \varepsilon(LWh)_n \qquad 2.13$$

Where ε is the unclogged porosity of the media; L is the wetland length (m); W is the wetland width (m); h is the water depth (m).

The theoretical or nominal detention time (τ_n) also refered to as the hydraulic retention time (HRT) is defined as the nominal wetland volume divided by the water flow and is given as:

$$\tau_n = \frac{V_n}{Q} = \frac{\varepsilon(LWh)_n}{Q} \qquad 2.14$$

Where τ_n is the nominal hydraulic detention time (days); Q = flow rate (m^3/d). It accounts for the fraction of the wetted section available for flow.

For a HSSF CW that hydrodynamically behaves as a plug flow reactor, zero mixing occurs and all flow will remain in the reactor for τ_n. However, deviations from plug flow conditions occur due to mixing and dead zones, resulting in the distribution of the residence time about τ_n.

The actual detention time (τ) of a CW is defined as the active volume divided flow rate and is given as:

$$\tau = \frac{V_{active}}{Q} \qquad 2.15$$

Where τ is the detention time (days); V_{active} is the CW volume invloved in the flow (m^3).

Dimensionless time (θ) can replace nominal detention time when the respose curves from different systems or measurements carried out at different times are to be compared and it given as:

$$\theta = \frac{t}{\tau_n} \qquad 2.16$$

Where θ is the dimensionless time; t is the elapsed time (days); τ_n is the nominal hydraulic detention time (days).

The volumetric efficiency (e_v) defines the fraction of the reactor volume that is actively involved in flow. It shows the actual volume that contributes to the flow compared to the theoretical conditions and is given as:

$$e_v = \frac{\tau}{\tau_n} = \frac{V_{active}}{V_{nominal}} \qquad 2.17$$

Where, e_v is the volumetric efficiency (dimensionless); V_{active} is the active wetland volume (m^3) $V_{nominal}$ is the nominal wetland volume (m^3)

The relationship between the HLR and the nominal detention time of a CW is an inverse proportion and is given as:

$$q = \frac{Q_i}{LW} = \frac{\varepsilon h}{\tau_n} \qquad 2.18$$

Where q, Q_i, L, W, ε and h are the hydraulic loading rate in m/d, flow rate in m^3/d, length in m, width in m, unclogged porosity and depth in water in m respectively.

The actual or interstitial velocity is given as:

$$v = \frac{Q}{\varepsilon A_c} \qquad 2.19$$

Where v is actual water velocity (m/d); Q is rate of flowof water through the system (m^3/d); εA_c is interstitial area perpendicular to the flow (m^2)

2.5.2 Tracer Experiments

Tracer tests are frequently used in hydrology of the surface and groundwater. They offer valuable insights into the hydraulic aspects of hydrological systems such as the functioning of CWs. Tracer experiments have been conducted to study mixing in wetlands, influence of substrate congestion on transport times in gravel beds, influence of pond properties on flow patterns, dependence of flow patterns on wetland geometry and testing of wetland models (Chang *et al*., 2011; Werner and Kadlec, 2000). Tracer tests have a rigorous mathematical basis and provide additional information about the subsurface. Tracer tests involve injecting a chemical tracer into the CW and following its recovery, over time, at various observation points.

According to Axelsson *et al*., (2005), three important aspects must be carefully considered before performing a tracer experiment, which are the trace material to be used, the amount of tracer for injection and the sampling plan (sampling points and frequency). The selected tracer must not be present in the system or must be much lower than the expected tracer concentrations (low background concentration). It must not react with or be absorbed by the media and must be easy (fast / cheap) to analyze, must be non-toxic, affordable, have good solubility in water, have negligible effects on transport properties (density, viscosity, pH etc.). Frequently used tracers in the CW field are salt tracers (iodide, lithium, bromide, chloride) and fluorescent dyes (fluorescein and rhodamine WT). Using salts as tracer material is common in the literature and can be an easy and convenient way of evaluating hydrodynamic of a CW (Chazarenc *et al*., 2003).

After tracer selection, the mass to be injected is determined. Factors such as the detection limit, the background of the tracer, the injection rate and the distance are all taken into account when estimating the required mass of the tracer. In general, tracer tests should be designed so that the concentration of the tracer reaches at least 5-10 times the detection limit (Axelsson *et al*., 2005). The sampling frequency should generally be quite high, but may decrease over time. In general it is better to collect too many samples than too little.

2.5.3 Residence Time Distribution

Danckwerts (1953) first extensively analyzed the residence time distribution (RTD) of reactors. Levenspiel (1972) further elucidated the theory of RTD. Since wetlands are indeed reactors, these basic principles from chemical engineering have been applied to them. RTD is evaluated by measureing the time a tracer material injected at the inlet of a wetland spends in the system.

the generally determined by introducing a tracer impulse and measuring the time spent by the tracer in the interior of the wetland. Residence time is a distribution and therefore important parameters such as the actual residence time and dispersion can be determined from the calculation of the different central moments (Kadlec, 1994). For a pulse tracer injected in a CW, the RTD function is given as:

$$E(t) = \frac{Q(t)C(t)}{\int_0^\infty Q(t)C(t)dt} \qquad 2.20$$

Where E(t) is RTD function (d^{-1}); C is concentration at the outlet (mg/l); Q is volumetric water outflow rate at time t (l/d); t is time of sampling (days).

The recovered tracer mass is the zeroth moment and is given as:

$$M_0 = \int_0^\infty Q(t)C(t)dt = \sum_{i=1}^{n} Q_i\, C_i \Delta t_i \qquad 2.21$$

The first moment, which represents the actual tracer residence time (τ, in days), defines the centroid of the tracer response curve and is given as:

$$M_1 = \tau = \int_0^\infty tE(t)dt = \frac{\sum_{i=1}^{n} t_i Q_i C_i \Delta t_i}{\sum_{i=1}^{n} Q_i C_i \Delta t_i} \qquad 2.22$$

The second moment corresponds to the variance (σ^2, in days^2). It characterizes the spread of the curve about the centroid. This spread is caused by the heterogenous nature of the system which causes preferential flows. Therefore, t^2 is replaced by $(t-\tau)^2$ to give the dispersion and it is given as:

$$M_2 = \int_0^\infty (t-\tau)^2 E(t)dt = \sigma^2 \qquad 2.23$$

The number of tanks in series (N) is estimated as (Kadlec and Wallace, 2009):

$$N = \frac{\tau^2}{\sigma^2} \qquad 2.24$$

The hydraulic efficiency of a CW, which is index of the hydrodynamic conditions of a system is calculated as:

$$\lambda = e\left(1 - \frac{1}{N}\right) = \frac{\tau}{\tau_n} = \frac{V_a}{V_n} \qquad 2.25$$

where e is effective volume; τ is mean tracer detention time (days); τ_n is nominal hydraulic residence time (days), V_a is active volume (m^3)and V_n is nominal or design volume (m^3).

The dimensionless variance (σ_θ^2) of the tracer is given as:

$$\sigma_\theta^2 = \frac{\sigma^2}{\tau^2} \qquad 2.26$$

The relationship between σ_θ^2 and N is given below:

$$\sigma_\theta^2 = \frac{1}{N} = \frac{\tau - \tau_p}{\tau} \qquad 2.27$$

The Peclet number (*Pe*) is a measure of how close the flow is to either plug or mixed flow an is given as:

$$Pe = \frac{uL}{D} = \frac{1}{P_D} \qquad 2.28$$

Where with u is the characteristic velocity; L is the characteristic length; D the characteristic diffusion coefficient in the direction of fluid movement; and P_D is the wetland dispersion number.

When Pe tends to infinity, there's no mixing and the flow pattern is ideal plug-flow, when Pe tends to 0, the flow pattern is completely mixed-flow. The relationship between σ_θ^2 and Pe is given as (Fogler, 1992):

$$\sigma_\theta^2 = \frac{2}{Pe} - \frac{2}{Pe^2}\left(1 - e^{-Pe}\right) \qquad 2.29$$

The shows that the dispersion of flow can also be detemined from the central moments of the tracer response curve. However, the direct determination of Pe from Equation 2.29 is not easy, as such iterative procedures are employed for its determination. To do that an initial Peclet number is chosen, and the equation solved for the dispersion number. The Peclet number is varied until the right hand side is equal to the σ_θ^2. The dispersion number is inversely proportional to the Peclet number.

2.5.4 Residence Time Distribution Models in Constructed Wetlands

A number of approaches have been applied in the characterization of flow patterns in HSSF CW. These approaches that are based on concepts developed in chemical engineering to characterize flow patterns in chemical reactors (Kadlec, 1994; Levenspiel and Turner, 1972) are often applied in the CW field. Two conceptual ideal models of CW hydrodynamics consider the system to be either complete stirred (CSTR) or plug flow (PFR) regimes. From numerous studies it is clear that CWs are neither plug flow nor well mixed, so that a non-ideal flow such as the Tanks In Series (TIS) and Plug Flow with Dispersion (PFD) have been used. These approaches are briefly discussed below:

- **Completely Stirred**

In a completely stired reactor (CSTR), which is an ideal reactor, the tracer materials are immediately and evenly disperser throughout the volume of the reactor (Levenspiel and Turner, 1972). In a perfectly mixed reactor, the starting composition is identical to the composition of the material in the reactor, which is a function of the residence time and the reaction rate. As the flow continues to enter the tank, water contaminated with tracer is displaced, resulting in a decreasing tracer output curve with a long tail.

- **Plug Flow**

Fluid passing through a Plug Flow Reactor (PFR) can be modeled as flowing through the reactor as a series of infinitely thin coherent "plugs". All liquids pass through the reactor in one file, with each plug having a different composition than before and after. PFR is an ideal flow reactor with no internal mixing of fluids. It assumes that all particles spends the same time in the reactor before exiting and this time is usually equal to τ_n, the nominal detention time. Long and narrow tanks exhibit plug flow behaviour because of very liminted dispersion in the system (Metcalf and Eddy, 1991). A tracer material injected at the input end of a CW will also exit the system without mixing and thus the response is a pulse output at τ_n.

- **Tank In Series**

Several studies on CW hydrodynamics indicate that flow patterns by such systems are not ideal. The hydrodynamic behaviour of subsurface flow CWs is said to be somewhere between completely stirred and Plug Flow. It is already established fact the conditions in most CW beds are not homogenous, thus very signficant levels of dispersion and mixing can occur as water moves through the substrate. Different flow models have been applied to the characterization of real-world flows. The *TIS* model, which assumes that a CW can be represented as a set of *CSTRs* of equal volumes, is considered state-of-the-art because of the proximity of fit that is reportedly reached. The number of completely stirred tanks in series can be any integral number from 1 to ∞. The typical response curves for a *TIS* reactor are shown in Figure 2.11.

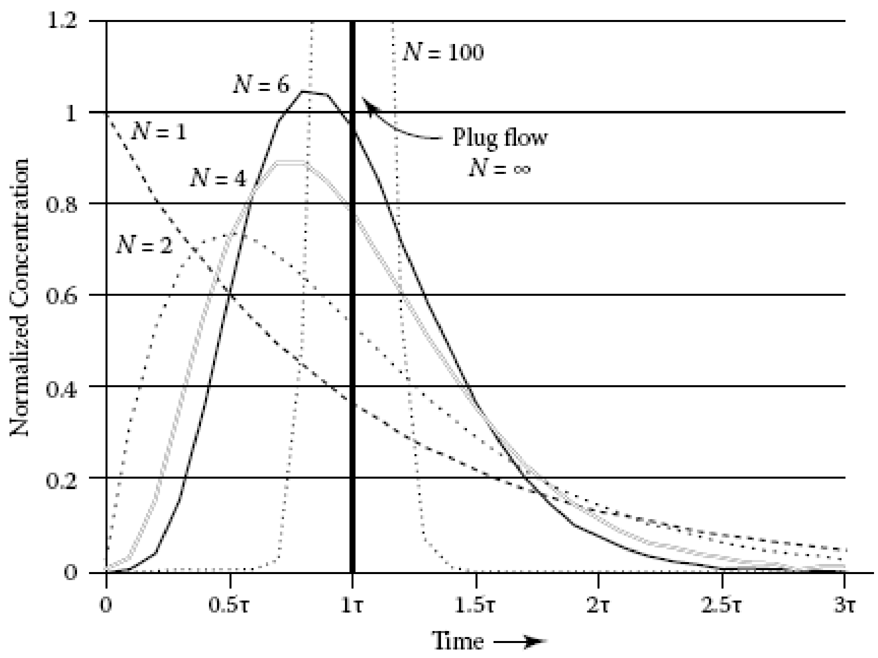


Figure 2.11 TBC for tanks in sries (Kadlec and Wallace, 2009)

Figure 2.11 illustrates the shape of the tracer response curves when N tends from 1 to ∞. If N is equal to 1, the CW behaves like a *CSTR* and, if N is equal to ∞, it acts as a *PFR*. This behaviour and the associated mathematics are well documented (Fogler, 1992, Levenspiel, 1972). Models based on the *TIS* hydraulics easily capture dispersion in a CW, as well as the existence of preferential flow paths under different hydraulic efficiencies (Kadlec and Wallace, 2009). With less complexity, models based on the *TIS* hydraulics are regarded as the most appropriate for describing flows in CWs (Chang *et al*., 2011).

- **Plug Flow with Dispersion**

The *PFD* reactor is another non-ideal flow reactor. Following a pulse injection of a tracer is into a flow that can be characterized as turbulent, the tracer materials move by convection and also spread in different directions. This spreading also known as dispersion is caused by a combination of diffusion as a result of turbulence in the system, molecular transfer from zones of high concentration to zones of low concentration and velocity gradient in the system. Data fitting are mostly used in describing PFD reactors as direct analytical solutions are very difficult.

According to Fogler (1992), the most appropriate baundary condition for such reactors is the *closed-closed* boundary condition, which implies that particles cannot move back once they have entered the inlet or outlet pipes.

2.5.5 Residence Time Distribution Model Applications to Constructed Wetlands

The hydraulic behaviour of various wetland systems have been extensively studied and different models have been used to simulate such behaviour.

Giraldi *et al*., (2009) analyzed the hydrodynamics of a vertical flow CW of 33 m^2 treating secondary municipal effluent to evaluate three levels of saturation conditions. Tracer tests were performed with rhodamine WT. BTCs were analyzed using classical RTD analysis and a numerical plug flow model with longitudinal dispersion. The study concluded that mixing was affected by water content.

Kadlec, (1994) studied the detention and mixing in a FWS CW using tracer test with lithium as the tracer material. Water flow in the system was found to be between plug and completely stirred. The *PFD*, *TIS* and a series-parallel network of tanks were used to determine the exit tracer concentration. They concluded that all models were able to fit the concentration curves of the exit tracer, but that the network model fitted better with internal measurements.

Chang *et al*., (2011) carried out a tracer test using RWT to evaluate the relationship between hydraulic retention time and transport processes in an on-site HSSF CW. They modeled the hydraulic behaviour of the system with the *TIS* model and concluded that coupling the HRT with the *TIS* model can provide good information on maintaining the needed processes for nitrogen removal across different parts of the system.

Garcia *et al*., (2004) modeled the hydraulic behaviour of a pilot HSSF CW using different substrate materials and various length to width ratios. The *PFD* and *TIS* were applied. They stated that the *TIS* model was a more accurately representation of the flow in the systen than the *PFD* model, although the *PFD* gave a better tracer response. They reported that the higher the aspect ratio of a CW, the better the hydraulic behaviour of the system and also the lower the internal dispersion.

Chazarenc *et al.*, (2003) modeled the RTD of a HSSF CW in France using the *PFD* and *TIS* models. Six experimental RTDs were monitored and modelled and it was observed that the design did not allow for adequate mixing but gave rise to areas of higher flow intensity. They concluded that the conceptual models could fit all response curves.

Werner and Kadlec, (2000) used a network of TIS model on plug flow channels to evaluate flow of water in a CW following 47 tracer experiments. They stated that the model realistically reproduced the the residence time distribution of the system and concluded that their approach was more flexible than TIS model and thus better suited for the description of various anomalies in the system.

Albuquerque and Bandeiras, (2007) studied the hydrodynamic characteristics of HSSF CW using trace techniques, with respect to factors that influence transport of particles in the system. Data fitting of the advection-dispersion-reaction model and moment method were employed in examining the magnitude of longitudinal dispersion. Flow characteristics and the existence of dead zones were estimated by data fitting of the TIS model. They reported that macrophyte development had little influence on the longitudinal dispersion of the system and also that the flow regime in the system was a plug flow. They concluded that *TIS* better explained the tracer response and that the moment method underestimated the Paclet number compared to the results obtaine by data fitting.

Bodin *et al.,* (2012) studied the effects of inlet design and vegetation type on tracer dynamics and hydraulic performance of eighteen experimental FWS wetlands using lithium chloride as tracer. Residence time distribution was calculated using moment method and a novel Gauss modeling approach to evaluate key hydraulic parameters such as active wetland volume, dispersion and hydraulic efficiency. They concluded that compared to the moment method, the Gauss modeling provided more reliable hydraulic efficiency but less reliable number of tanks in series.

Maloszewski *et al*., (2006) investigated the hydraulic parameters in three parallel gravel beds in Poland using tracer experiments. The system was modeled with the multi-flow dispersion model. They stated that the use and calibration of the multi-flow dispersion model for CWs was possible but that the obtained tracer characteristics, particularly for dispersion, which suggested that zones of stagnation played no significant role in the CW, were not realistic.

2.6 CFD Modeling of Constructed Wetland Hydraulics.

As modern computing technology continues to advance and also because of the identified limitations of the conventional tracer techniques for hydrodynamic evaluation, the application of CFD technology has been recommended as an alternative method fluid flow and sediment transport evaluation. CFD is a branch of fluid dynamics providing a cost-effective means of simulating real flows by the numerical solution of the governing equations. The Navier- Stokes and continuity equations are the foundation of CFD. Yan (2013) gave a summary of the advantages of CFD utilization for CW hydraulics evaluation:

- It allows for the numerically simulation of the hydrodynamics of CWs with complex geometries, and give results that can be very difficult to achieve experimentally or with the other conventional modeling techniques.
- It provides an alternative approach for the evaluation and analysis of the hydraulics of existing wetlands.
- It is an important design tool as it can be used for the prediction of the hydrodynamic characteristics of a CW, as well as for the identification of flow patterns and short-

circuiting challenges before cnstruction, unlike conventional desin approaches that do not take into consideration the feature hydraulic problems. It can also be used to study the effect of different modifications such as the addition of baffles or inlet and outlet pipe repositioning.

Studies have evaluated the hydrodynamic behaviour of CWs using two and three dimensional modeling. Rangers *et al*., (2016) modelled the effect of flow in a CW on pollutant removal using CFD and reported that the length of the CW had the most effect on it's the hydraulic efficiency. They also observed that incorporation of baffles significantly influenced the system hydraulics. Fan *et al*., (2008) also modeled the effect of wetland design on the flow of water in a subsurface flow constructed wetland using CFD. The residence time distribution was evaluated using the particle trajectory model in Fluent 6.22, and the result revealed that the configuration of the CW significantly influenced the system performance. The hydraulic dead-zone and particle removal efficiency in the base frame of a constructed wetland was evaluated by Choi and Park, (2013) using CFD. The fraction of hydraulic dead-zone was attributed to the artificial island development in the CW. They concluded that experimental trends in the HRT variation could be identified with CFD analysis. Engstrom *et al*., (2010) modeled bacterial transport and removal using first-order kinetic equation coupled to CFD model. Figure 2.12 shows the simulation results with the surface plot showing E. coli concentration and streamlines representing the velocity field after 1 week.

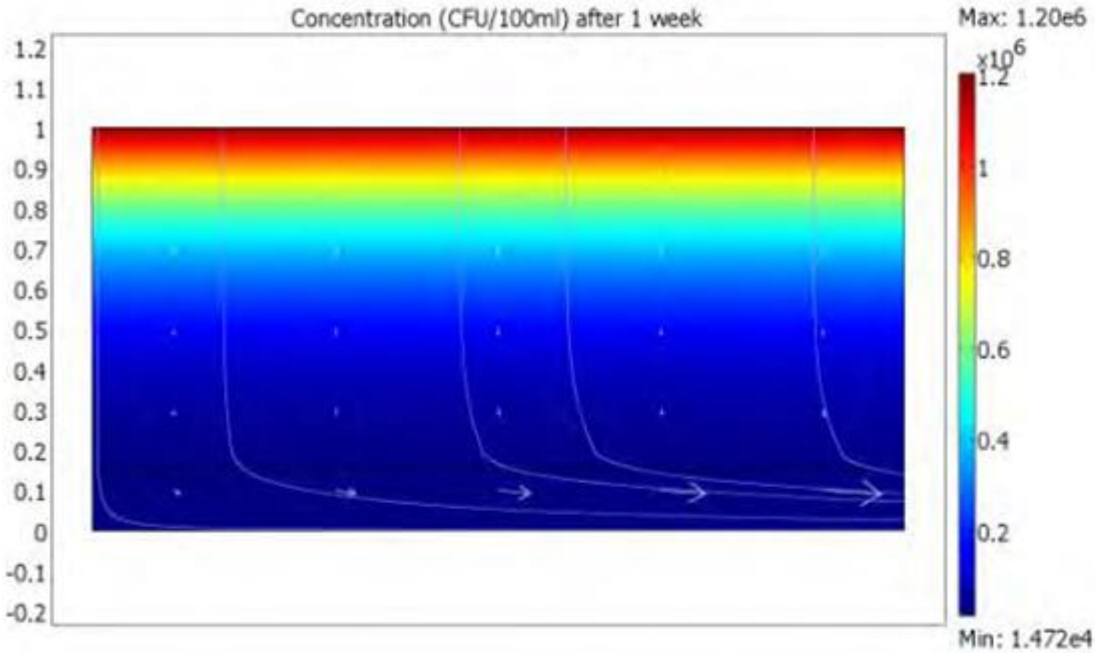


Figure 2.12 2D modeled concentration and velocity field in a CW (Engstrom *et al*., 2010)

2.7 Summary of Literature Review/Research Gap

The uncontrolled discharge of wastewater from slaughterhouses in Nigeria has been identified as one of the major sources of environmental degredation because they pollute both surface and underground waters, rendering them infit for human consumption and utilization for other purposes, and thus has very severe consequences, not only for human existence, but also for ecological sustainability. CW has been identified as a viable option for slaughterhouse wastewater treatment. The technology has evolved in developed world from the application of the basic concepts to more complex hybrid and integrated systems. Despite the numerous publications on CWs over the past decades, and its recognition as a viable technology for treating wastewater, there is a notable gap in literature regarding its potentials for slaughterhouse effluent bioremediation in developing countries, and studies evaluating CWs performance for of slaughterhouse wastewater treatment are fairly limited both in the temperate and the tropical climates. There is little or no published literature on application of CW for slaughterhouse wastewater treatment in Nigeria.

Wetland substrates support the wetland vegetation, provide sites for biochemical and chemical transformations, and provide sites for storage of removed pollutants. Gravel is the most common media used in CWs. Very important is the fact that substrate materials must be cheap and also locally available for easy implementation of CWs. Recently, studies of the use on non-conventional materials as CW substrates have increased. Several studies have investigated different types of organic solids (corn cobs, green waste, wheat straw, softwood and hardwood) as alternative wetland media (Cameron and Schipper 2010, Tee *et al*., 2009). Most of the studies on the feasibility of non-conventional substrates were performed on experimental or pilot scales, and some using synthetic wastewater. Treatments in experimental or lab-scale systems do not adequately describe the processes that occur in field-scale systems, due to factors like significant edge effects and synthetic wastewater cannot be compared to real wastewater, which is much more complex (Kadlec and Wallace 2009). There are also lab-scale studies on the efficacy of PKS as a wetland substrate (Chong et al., 2009; Jong and Tang, 2015) with varying degrees of the PKS processing prior to use, which may not be achievable in large-scale systems. Studies on PKS capacities in real-life CWs, its effective lifespan in such a system and its influence on water flow in a CW are lacking in wetland literature. Nguyen *et al*., (2013) in their review of the applicability of agricultural waste and by-products for the sequestration of heavy metals from wastewater, stated that agricultural waste has shown equal or even increased adsorption capacities compared to conventional material, but emphasized the existence of various gaps that require further investigation, including assessing the performance of agricultural waste and by-products under real wastewater systems. There is need to supplement these gaps.

Also, the knowledge that statistically different results have been obtained for different macrophyte species performance raises the interest in experimenting with more macrophytes. Most studies on CW performance were performed using the system as polishing units after secondary treatment. The literature on emergent macrophytes for CWs is quite extensive (Burke, 2011, Baskar *et al*., 2014, Dipu *et al*., 2010) but most of the information concerns the tertiary treatment of sewage. Studies on macrophytes survival and growth in CWs for secondary treatment of slaughterhouse effluent is very limited. *Typha spp* and *Phragmites spp* have been extensively evaluated for use in CWs, but there is little documentation on the use of other potential macrophytes such as *Colocasia Esculent* and *Thalia Geniculata* as emergent macrophytes for CWs.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Description of the Study Area

The research was confined to Anambra State due to funding constraints, but its recommendations will be applicable to other states in Nigeria. Ecologically, the state falls within the rainforest zone of Nigeria. Monthly normal climate data (2006-2015) from the NIMET Synoptic Station, Awka indicates that temperatures reach a maximum of 35.7℃ in February and a minimum of 20.4℃ in January. The average annual rainfall in the study area is 1923.7mm. Most of the precipitation falls from April to October, and the area is very dry from December through March. The average annual potential evapotranspiration in the study area was calculated as 1965.8mm using the Thornthwaite method.

Seven slaughterhouses out of the major ones in Anambra State that dealt mainly on beef were selected for the wastewater characterization and treatability studies. The seven slaughterhouses were randomly selected to ensure that the major cities of Anambra State were covered and at least two from each senatorial district. The slaughterhouses are shown in Figure 3.1, and their coordinates are: the Umunya slaughterhouse in Oyi L.G.A (6.207611667^{o}N and 6.905594^{o}E); Nkwo-Nnewi slaughterhouse in Nnewi North L.G.A (6.019105^{o}N and 6.90853^{o}E); Amansea slaughterhouse in Awka-North L.G.A (6.248326667^{o}N and 7.136735^{o}E); Eke-Ekwulobia slaughterhouse in Aguata L.G.A (6.018026944^{o}N and 7.080091^{o}E); Agulu slaughterhouse in Anaocha L.G.A (6.093185^{o}N and 7.031785^{o}E); Eke-Awka Etiti slaughterhouse in Idemili South L.G.A (6.03555^{o}N and 6.962635^{o}E); and Ochanja slaughterhouse in Onitsha South L.G.A (6.133826667^{o}N and 6.785008 ^{o}E). They varied in size from small private facilities to large municipal ones, and they all had approval from the local government authorities. The Agulu slaughterhouse, which was the selected slaughterhouse for the on-site bio-remediation system implementation, is in Agulu town. The town has one of the largest populations in the state. Its cordinates are latitudes 6.04^{o}N and 6.09^{o}N and longitudes 7.00^{o}E and 7.03^{o}E, and it lies within

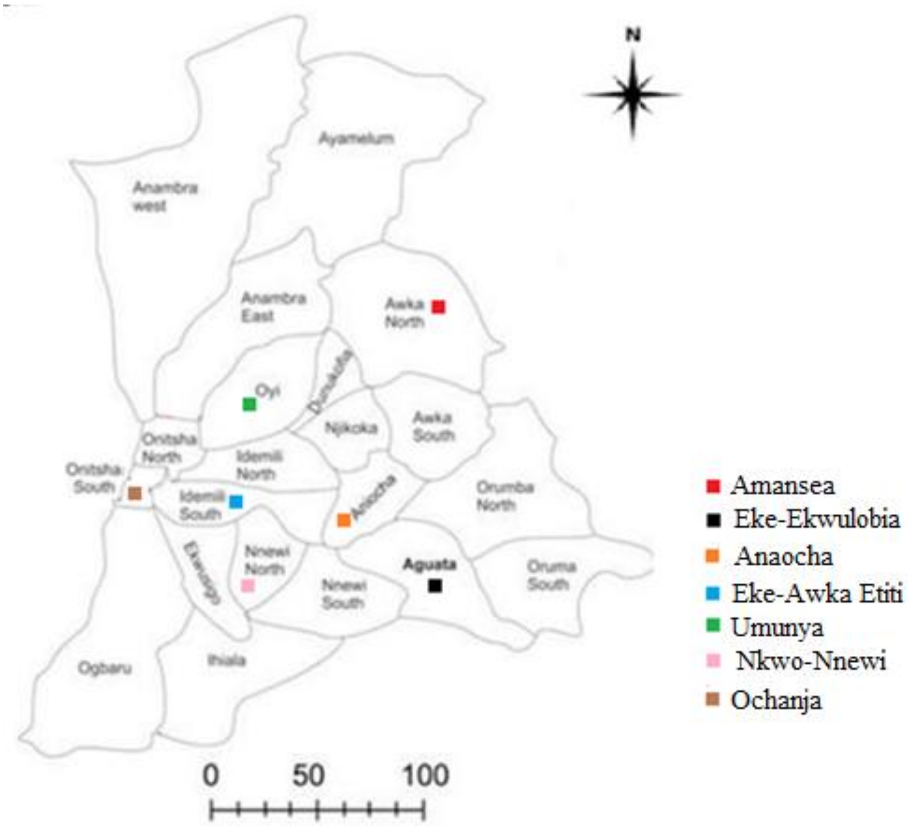


Figure 3.1 Map of Anambra State showing the study areas

the boundaries of the Capital Territory of the State. It lies within latitudes 6.04^{o}N and 6.09^{o}N and longitudes 7.00^{o}E and 7.03^{o}E. It covers an area of about 85km^2 and shares boundaries with eight towns namely: Nise in the North, Mbaukwu in the North-East, Awgbu in the South and South-East, Nanka in the South, Nri in the North-East, Adazi to the West, and Obeledu and Agulu Uzoigbo to the South-West. Agulu lies along the main road that links the capital city (Awka) with Ekwuluobia and further to Imo State. The slaughterhouse is located in Obeagu village, along the Adazi-Agulu-Agulu Uzoigbo link road and serves as a major meat source for the surrounding towns. The total area of the slaughterhouse is around 1 hectare.

The summary of the experimental procedure is show in Figure 3.2

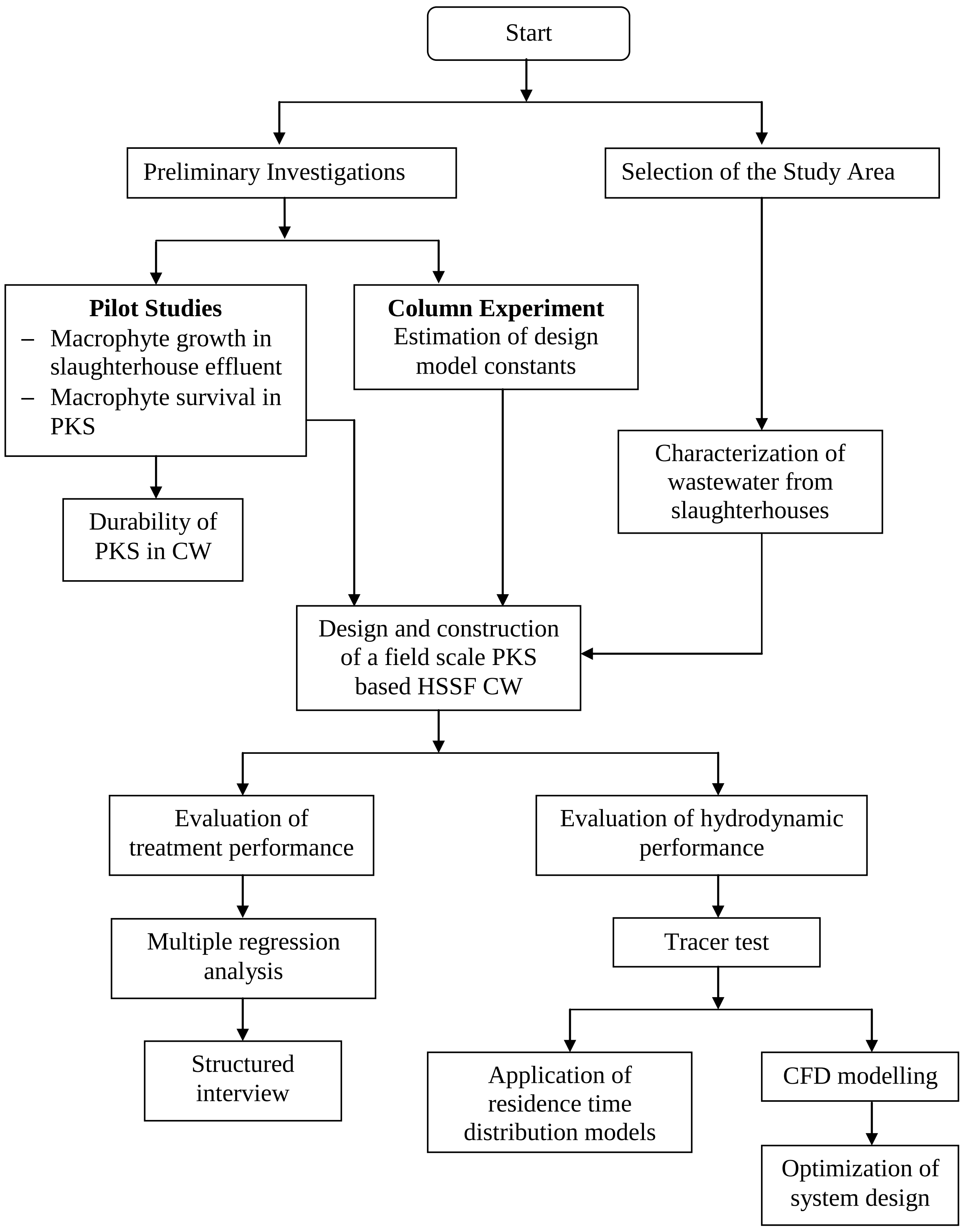


Figure 3.2 Summary of the experimental procedure

3.2 Wastewater Characterization Study

3.2.1 Sampling and Data Collection

Two wastewater samples were collected monthly from October 2016 to December 2016 at the seven studied slaughterhouses. The samples were collected in the morning or early afternoon from the major channels of effluent outflow. This was because meat processing and slaughterhouse cleaning were carried out in the morning or early afternoon. Sample collection was done using 500 ml plastic containers. The containers used for sample collection were cleaned by washing in non-ionic detergent and rinsing with tap water prior to usage. During sampling, field measurements were first carried out for pH, EC, Temperature, TDS, NO_3-N, NO_2-N, and PO_4^{3-}. The plastic containers were then rinsed with sample wastewater three times and filled to the brim. The samples were immediately transported to the laboratory in ice block filled cooler and stored in the refrigerator at about 4^OC prior to analysis of other physicochemical parameters. During the three months period, investigations were also carried out to estimate daily wastewater production. Average daily wastewater production was estimated by dividing the water storage tank capacities by the number of days of usage.

Facilities at the slaughterhouses, such as water supply, electricity, rail system, cold room etc., were compared to what was obtainable in standard abattoirs in terms of presence and functionality. At a slaughterhouse where a facility was present and at the same time functional, a grade of 1 was assigned. Where a facility was present but not functional, a grade of 2 was assigned. Slaughterhouses where a facility did not exist were also noted. Wastewater collection and treatment facilities at the seven slaughterhouses were also evaluated based on presence and functionality.

3.2.2 Data Analysis

The treatability of the wastewater from the seven slaughterhouses was evaluated using the Biodegradability Index (B.I), which is a crucial step before biological wastewater treatment technology can be implemented. The index, which is the BOD to COD ratio, has been used as one of the well-adopted surrogates for biodegradation capacity. It serves as a benchmark to determine if wastewater is biodegradable or not (Abdalla and Hammam, 2014). If the ratio of BOD to COD is greater than 0.6, then the effluent is considered to be suitable for biological treatment. If the ration is in the range of 0.3 and 0.6, then additional measures must be taken, such as seeding with microorganisms, before biological treatment can be carried out. This will likely slow down the treatment process, as it takes time for the microorganisms that are involved in the degradation process to acclimatize. If the ration is less than 0.3, then the wastewater cannot be treated by biological means, as the wastewater may contain pollutants that are toxic to the microorganisms and thus inhibits the metabolic activity.

3.3 Pilot Studies

3.3.1 Experimental Setup

Six rectanglular wetland cells made of transparent plastics (Figure 3.3) were setup as the pilot horizontal subsurface flow CWs at the grounds of the Agulu slaughterhouse. Each had a depth of 0.3m, length of 0.5m and width of 0.35m giving a volume of 0.053m^3. To evaluate the growth and treatment response of locally available macrophytes in slaughterhouse wastewater, four of the pilot wetlands were used, while the remaining two pilot wetlands were used to evaluate the performance of PKS as a HSSF CW substrate.

Figure 3.3 Transparent plastic containers that served as the pilot HSSF CW cells

PKS samples were collected from the Nkwo-Agu oil mill, Adazi-Nnukwu, Anambra State and sun dried for 2 days to remove unwanted insects. The samples were then minimally washed to eliminate unwanted soil particles. This can be avoided by ensuring the substrate is sand free. No further processing was carried out on the PKS (rinsing in hot water to remove any oil residue left after washing) as this may not be feasible if large quantities of PKS are to be used for a field-scale system. It was then sieved to produce substrates of >5 mm size. Gravel samples of size 5-12 mm were procured from a supplier in Awka, Anambra State. The samples were washed thoroughly to remove mud and sand particles.

Three locally available macrophytes selected for the study were *Thalia geniculata* (Fire Flag/ Alligator Flag), *Colocasia esculenta* (Green Taro Plant) and *Typha latifolia* (Cattail). The macrophytes were identified using the Plant Directory of the Center for Aquatic and Invasive Plants (CAIP), of the Institute of Food and Agricultural Sciences, University of Florida. Besides having high potential productivity, deep rhizomes and root systems, and are readily cultivatable, the three plants where chosen because they were widely distributed in the study area. *Typha latifolia* are herbaceous, perennial plants with long, slender green stalks topped with brown, fluffy, sausage-shaped flowering heads. They grow in or near water, in marshes, ponds, lakes, depressions and can grow to a height of 2-3 m. They are aggressive invaders and their rhizomes spread horizontally beneath the surface to start new upright growth and they also reproduce by seed. *Typha latifolia* has been found to be tolerant of water level fluctuations and moderate soil salinity. *Colocasia esculenta*, also referred to as "Elephant Ear" is a tropical perennial herbaceous aquatic plant with heart-shaped, dark green leaves. They can grow up to 1-2 m tall. Tubers are spherical and about the size of tennis ball often covered with brownish skin and hairs. Each plant grows one large tuber often surrounded by several smaller tubers. *Thalia geniculata* are large upright plants. They grow to a height of 1-2.5 m and spread via short, thick underground rhizomes to form large clumps. The leaves have a long and thick stalk and a very large leaf blade (up to 75 cm long and 25 cm wide). They also tolerate water level fluctuations and moderate soil salinity. Macropytes were collected from natural wetlands (Figure 3.4).

Figure 3.4 *Thalia geniculata* and *Colocasia esculenta* collection from natural wetlands in the study area

The *Typha latifolia* used for the study was obtained from a natural wetland in Anam, in Anambra West L.G.A. The *Thalia geniculata* and *Colocasia esculenta* were obtained from a natural wetland along the Awka-Enugu old road, close to the Awka wonderland, and a natural wetland in Amawbia, along the Awka-Ekwulobia road respectively. All plants were uprooted and the above-ground portion were cut at a height of 0.2 m for the *Typha latifolia* and 0.5 for *Thalia geniculata* and *Colocasia esculenta* and transported in water to the experimental site to avoid dehydration. The samples were then cleaned in flowing water to remove the soils attached to the root before being transplanted into the CWs. The macrophytes are briefly described below.

In May, 2016 three of the pilot wetland cells were filled with gravel and five healthy mature plants of *Thalia geniculata*, *Colocasia esculenta* and *Typha latifolia* were planted directly into the wetland cells giving a density of 28.6 shoots/m^2. One cell was filled with gravel and left unplanted to serve as control. The two remaining cells were filled with PKS and planted with

Thalia geniculata and *Typha latifolia* respectively. After planting, all the cells were filled with stream water to enable the plants acclimatize for two weeks, after which the plants were left to establish for ten weeks. Plants were grown outdoors and exposed to ambient environmental conditions. Figure 3.5 shows the cells after planting and after 2 months of establishment.

(a) **(b)**

Figure 3.5 Pilot wetland cells (a) after planting; (b) after 2 months of establishment

A plastic tank of 0.1 m^3 was provided to serve as feed tank. The feed tank was connected to the pilot cells by simple pipe network with a control tap to avoid turbulence when filling the cells. Prior to each experimental treatment, the feed tank was manually filled with slaughterhouse wastewater, sealed to encourage anaerobic conditions and allowed to stand for 24 hours for primary treatment. The level of water in cells was kept at 0.02 m below the surface. Perforated PVC pipes was installed in each cell for sampling and plastic taps were installed at the bottom of the cells for washout.

3.3.2 Operational Procedure

The pilot wetland cells were loaded and drained on a batch mode, i.e. one off feed for the whole retention time of an experimental set from August 2016 to February 2017. Davison *et al*., (2005) observed that little additional removal occurs after 10 days in warmer climates. Also Stein *et al*., (2006) reported a rapid decline in organics concentration within the first three days and residual levels approached by day 14. Therefore, the feed cycle used was based on a retention time of 14 days and was maintained throughout the study period. The batch loading format had the advantage of increasing oxygen transfer into the exposed pore spaces and roots, and thus leading to an increase in the level of treatment attained especially for oxygen demanding processes. The sequence followed on each loading day was: reading the water level in the cell; emptying the cell completely; and refilling with wastewater from the head tank. Loading was done between 7.00am and 9.00am.

3.3.3 Data Collection and Analyses

Influent and effluent samples were analyzed to determine the treatment performance of the macrophytes and PKS substrate in the pilot HSSF CW. Influent samples were collected during wastewater loading, while effluent samples were collected during draining. The samples were also collected using 500 ml plastic containers, labeled and transported immediately to the Springboard Research Laboratory, Awka for physicochemical analysis. Water level in the cells were periodically calibrated to determine influent volumes. The cells were allowed to stand for 10minutes before the calibration, to ensure that the bed was completely drained. To calibrate, a cell was emptied and filled with known volume of wastewater, while the level of water in the cell was recorded. Graphs and equations were obtained from the water level-volume calibration, for each measurement. Calibration was done once a month to account for changes in volume due to increasing rooting biomass. Transparent polyethylene film was installed on a wooden frame to prevent rainwater from entering the cells.

Plant growth in the pilot HSSF wetland was monitored by measuring the height and abundance in each cell. Shoot heights (from base to apex) of 5 tallest plants in each cell and standing shoot were measured every two weeks. Only shoots that were alive were recorded and counts were cross-checked by repeated counting. Macrophytes shoot density in the pilot wetlands were computed as:

$$Shoot\ Density = \left[\frac{Shoots}{Area\ (m^2)}\right] = \frac{Shoot\ Count}{Area} \qquad 3.1$$

All cells were harvested in November 2016 after 6 months growth. Plants were cut at the media surface. The harvested above-ground biomass were cut into fractions of 15 cm in length and dried at 100 ^oC for 24 hours before weighing to determine the dry weight.

The removal efficiencies of the pilot wetlands were presented in terms of mass removal rates and mass removal percentages, as they take into account the variations in the water volume due to ET losses. The mass removal rate (MRR) quantifies the pollutant mass per unit area removed by the system over a given period. It is given as:

Pollutant influent loading rate, ILR (g/m^2.d or g/m2.wk) = $\frac{C_i V_i}{A \times I}$ 3.2

Pollutant mass removal rate, MRR (g/m^2.d or g/m2.wk) = $\frac{C_i V_i - C_e V_e}{A \times I}$ 3.3

Pollutant mass removal efficiency (%) = $\frac{C_i V_i - C_e V_e}{C_i V_i} \times 100$ 3.4

Where C_i is the influent concentration (mg/l); C_e is the effluent concentration (mg/l); V_i is the influent volume (l); V_e is the effluent volume (l); A is the wetland surface area (m^2); I is the interval between wetland refilling (day or week or per batch)

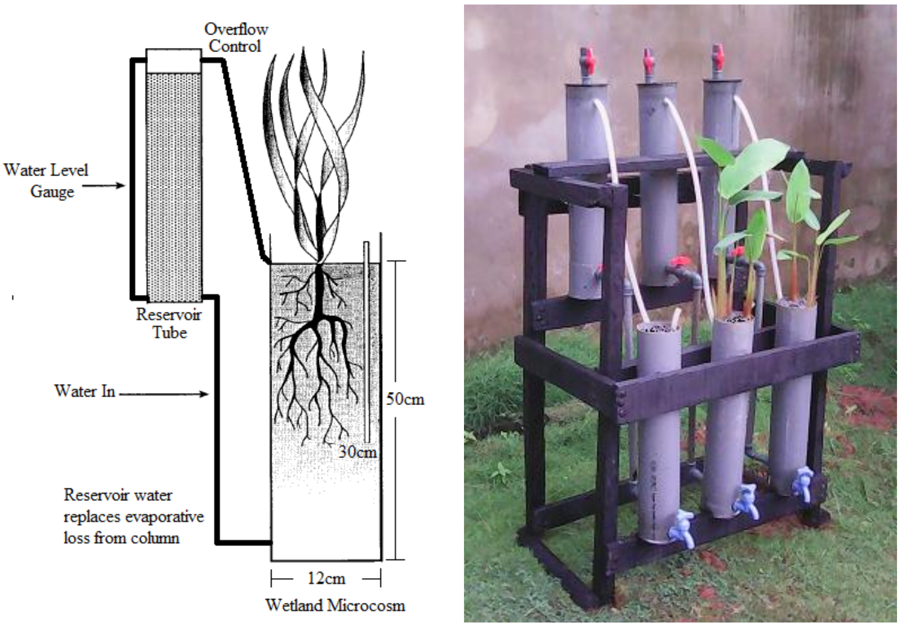
Evapotranspiration (ET) in each pilot wetland was obtained by subtracting the remaining water volume in the cell after each batch from the volume of water deduced after loading and dividing by the area of the cell. The daily ET rates (mm/day) were determined by dividing with the number of days between loadings.

Three properties of the PKS used in the pilot wetland cells were evaluated to determine their short-term durability as a HSSF CW substrate. This was done at the beginning of the pilot study

(May 2016), after 6 months of operation (November 2016), and finally after 20 months of operation (January 2018). The properties measured were the specific gravity, aggregate thickness and aggregate crushing value.

3.4 Column Experiment

Three HSSF CW columns were used for the study. The columns were constructed according to the procedures of Allen *et al.*, (2002). They were made of PVC pipes of height 60cm and diameter 12cm and the water depth was maintained at 50cm as shown in Figure 3.6.



(a) **(b)**

Figure 3.6 (a) Schematic diagram of column and water delivery system (Adapted from Allen et al., 2002) (b) Experimental setup

The three columns were filled with PKS. Two were planted with *Thalia Geniculata*, while the third was left unplanted to serve as control. Access tubes (1.1 cm diameter) were installed to a depth of 30 cm for sampling. Plastic taps were fitted on the floor of each cell for washout. The setup allowed for the water to be automatically kept at the desired level, with water lost to evapotranspiration replace immediately from the bottom of each column, thus each column functioned independently. To ensure no overflow and washout of the wastewater, a plastic tap was fitted at the inlet of the microcosm and was always closed prior to filling the reservoir from the top. The columns were left to acclimatize in stream water for 14days, and then allowed to establish in wastewater prior to experimental treatments.

Two 40 days batch incubations were conducted using the HSSF wetland columns from January 2017 to March 2017. The dry season experimental run was to allow for accurate sampling because there was no dilution effect through rainwater mixing with the samples. Columns were gravity drained a day prior to the incubation. During the incubation the columns were filled with pre-settled wastewater. The reservoirs were intermittently refilled from the top with tap water to compensate for evaporative losses. Wastewater samples were collected from the HSSF CW columns on a non-continuous basis (Stein et al., 2006; Kurup, 2017, Biederman, 1999) at days 0, 2, 5, 8, 12, 21, 30 and 40 for the first incubation, and days 0, 3, 6, 9, 14, 30 and 40 for the second incubation. Field measurements were also carried out before grab sample collection for physicochemical analysis.

3.4.1 Fitting of *k*-*C** Design Model to Experimental Data

The model developed by Kadlec and Knight, (1996) (Equation 3.5) is a widely accepted design tool for CWs. However, for the model to be used for the design of the PKS based field-scale HSSF CW, the parameter values were estimated using data from the column experiment.

$$\frac{(C_e - C^*)}{(C_i - C^*)} = exp^{-kt} \qquad 3.5$$

Where C_e is inlet concentration (mg/l); C_i is outlet concentration (mg/l); C^* is residual concentration (mg/l); k is volumetric rate constant (d^{-1}); t is time (d).

The rate constant (k), an important term in the model is dependent on temperature. The effect of temperature on the rate constant was modeled using the modified Arrhenius relationship given as (Kadlec and Knight, 1996):

$$k = k_{20}(\theta)^{(T-20°)} \quad 3.6$$

Where k is the rate constant at temperature T, (d^{-1}), K_{20} is the rate constant at 20 °C, (d^{-1}), Θ is the dimensionless temperature coefficient.

Rate constant at 20 °C reference temperature (k_{20}), temperature coefficients (θ) and background concentration (C*) were found by fitting the model predictions to the measured concentrations in the HSSF CW columns, and minimizing the sum of squared error (SSQE). BOD, TSS, NH_4-N, NO_3-N and PO_4^{3-} concentrations obtained during the first 40days incubation were used to obtain the best fit estimates of the model constants. Parameter optimization was performed in Microsoft Excel® using Solver®. This method assumes that the best model parameters or the line of fit of the predicted concentration is obtained when the sum of squared residual is between the measured and a predicted concentration is lowest (Equation 3.7).

$$SSQE = \sum_{i=1}^{N}(C_{eff} - C_{predicted})^2 \quad 3.7$$

3.4.2 Validation of the Model Constants

To validate the estimated model parameters, simulations were conducted with the model constants obtained from the calibration process. The data obtained during the second 40days incubation were used for the simulation. The goodness of fit between the measured and model predicted concentrations was evaluated using coefficient of determination (R^2), which indicates the total variance in the data set, and sum of squared error (SSQE). Also, calculated wetland areas were compared for the parameter estimates and the universal values in the literature.

3.5 Field-Scale Studies

3.5.1 On-site Bio-remediation System Design

The concept of the bioremediation system in this study was to channel wastewater from the slaughterhouse to a septic tank for primary remediation, and then through a HSSF CW for secondary remediation before been discharged. The system is a two reactor system in series.

- **Septic Tanks Design**

The primary task of a septic tank is to provide quiescent conditions in other to retain solids by settlement and scum (fats, oils and grease) by floatation, and also for biodegradation of organics and nutrients. The design considerations for the septic tank were the hydraulic retention time, tank volume, and compartmentalization. The effluent flow values used for the design were: typical average daily wastewater volume of 0.16 m^3; typical maximum daily wastewater volume of 0.22 m^3; design daily volume (0.22 m^3 + 25% safety factor) of 0.275 m^3. The values were estimated from the preliminary study of wastewater production at the Agulu slaughterhouse.

Septic tanks are normally sized to provide a minimum retention time of 24 hours based on the tank volume being 50 to 65% occupied by scum and solids. Longer detention time may be taken into consideration depending on the desired effluent strength. For this study, the septic tank was sized to provide a minimum detention time of 24 hours at peak wet season flow and at 50% sludge accumulation. Because the slaughter slab and evisceration area were outdoors and exposed to rainfall, the potential supply of rainwater from their catchment to the system was obtained by multiplying the area of the slaughter slab, the runoff coefficient of concrete (0.8) and the rainfall intensity. The average daily rainfall was obtained by dividing the monthly values by the number of days. To determine the volume of the septic, daily inflow was multiplied by the hydraulic retention time. The calculated minimum septic tank volumes are presented in Table 3.1.

Table 3.1 Estimated results of monthly effluent flow and septic tank volume

Month	Design Wastewater Flow (m^3/d)	Average Daily Precipitation (m/d)	Area (m^2)	Runoff (m^3/d)	Inflow (m^3/d)	Septic Tank Volume (m^3)
Jan	0.275	0.000752	112.5	0.0677	0.34	0.68
Feb	0.275	0.00115	112.5	0.1035	0.38	0.76
Mar	0.275	0.002132	112.5	0.1919	0.47	0.94
Apr	0.275	0.00493	112.5	0.4437	0.72	1.44
May	0.275	0.008226	112.5	0.7403	1.02	2.04
Jun	0.275	0.008633	112.5	0.7770	1.05	2.10
Jul	0.275	0.008245	112.5	0.7421	1.02	2.04
Aug	0.275	0.008458	112.5	0.7612	1.04	2.08
Sep	0.275	0.01113	112.5	1.0017	1.28	2.56
Oct	0.275	0.00759	112.5	0.6831	0.96	1.92
Nov	0.275	0.001547	112.5	0.1392	0.41	0.82
Dec	0.275	0.000219	112.5	0.0197	0.29	0.58
Mean	0.275	0.005251	112.5	0.4725	0.75	1.50

The maximum inflow into the septic tank was estimated at 1.28m^3/d in the month of September. Thus, the septic tank minimum volume was 2.56m^3. Septic tank geometry is an important consideration that affects residence time of solids in the tank. For this study, commercially available cylindrical plastic tanks were considered for the septic system, not only to reduce the construction costs, as they were cheaper than reinforced concrete tanks, but also due to the temporal nature of the study. For enhanced suspended solids and organics removal, a two compartment septic tank system was adopted with a volume ratio $V_1:V_2 = 1:1$.

- **PKS based Field-Scale HSSF CW Design**

The presumptive design method, which assumes that a certain amount of pollutant was removed by the primary treatment and uses the remaining contaminant concentration to size the system, was used in this study due to paucity of data on the treatment performance of septic tank systems as primary treatment units for slaughterhouse wastewater in Nigeria. Most HSSF CWs are designed for BOD removal. However, there are recommendations for wetland sizing to be based on the parameter that requires the largest footprint for treatment, which is considered the limiting design parameter (LDP). This parameter controls the dimensions of the wetland to ensure effluent target for all the parameters of interest are met (Reed et. al., 1995a). For this study, the wetland was sized for BOD, TSS, NH_4-N, NO_3-N and PO_4^{3-} removal.

Maximum raw wastewater parameter values obtained from the Agulu slaughterhouse during the characterization study were: BOD = 736.2 mg/l; TSS = 586.2 mg/l; NH_4-N = 92 mg/l; NO_3-N = 61 mg/l and PO_4^{3-} = 14 mg/l. These values were taken as the maximum strength of wastewater that will be discharging into the septic tank from the slaughterhouse. The removal efficiency of septic tanks treating different types of wastewater in the literature range from 50 - 70% for organics, 40 - 80% for solids and 30 - 65%, for nutrients (USEPA, 2000; Rahman *et al.*, 1999). For this study, conservative estimates of 50% organics, 40% solids and 30% nutrients removal were used. Therefore, the influent wastewater concentrations for the HSSF CW design were: BOD = 368.1 mg/l; TSS = 351.72 mg/l; NH_4-N = 64.4 mg/l; NO_3-N = 42.7 mg/l; PO_4^{3-} = 9.8 mg/l. The wetland was designed to meet values prescribed by FEPA (1991)/NESREA for effluent discharge in Nigeria. The target effluent concentrations for the design were: BOD = 50 mg/l; TSS = 30 mg/l; NH_4-N = 10 mg/l; NO_3-N = 20 mg/l; PO_4^{3-} = 5 mg/l.

The system was sized using the modified first order kinetic model (Equation 2.7, Chapter 2). The rate constants and temperature coefficients estimated for contaminants removal in PKS column experiment, as well as the lowest values of the irreducible background concentrations obtained during the preliminary experimental run were used in the design, except for NH_4-N which was found to be significantly higher than the theoretical background of zero. These parameter values are shown in Table 3.2.

Table 3.2 *k*-*C** Model Parameters for Sizing the PKS based HSSF CW

Parameter	$K_{20}(d^{-1})$	θ	C* (mg/L)
BOD	0.6042	0.9948	23.0
TSS	0.6236	1.0928	25.6
NH_4-N	0.2783	1.0505	0
NO_3-N	0.3235	1.016	0.36
PO_4^{3-}	0.3065	0.9537	0.42

Because the value of temperature coefficient for BOD was counterintuitive and contradicted values reported in the literature, system design for BOD removal was not adjusted for temperature as recommended by Dotro *et al.*, (2017).

Water depth is a very important system design, operation, and maintenance parameter. There is no consensus as regards the appropriate depth of substrate for HSSF CWs in the literature. Depths of between 0.4 m and 0.6 m are considered very effective (USEPA, 1999). From the pilot study on the growth and performance of macrophytes, it was established that both the *Thalia Geniculata* and *Colocasia Esculenta* roots and rhizomes penetrated more than 0.3 m into the substrate. Therefore, a bed depth of 0.5 m was selected for the system.

The calculation results are presented in Appendix B. Based on the average inflow of 0.75 m^3/d, NH_4-N required the largest area of 17.21 m^2 for treatment, which was set as the area of the HSSF CW. The average inflow was used instead of the maximum inflow to minimize the need for supplemental water in the dry season when the inflow was at its lowest, and also because of the fact that the actual slaughterhouse wastewater design inflow of 0.275 m^3/d was well below the average wetland inflow and the additional inflow from rainwater did not contribute significantly to the pollutant load.

Theoretically, longer channels are considered more likely to generate flows that are closer to plug flow, than wider channels. Good general ratios are considered to be 3:1 or 4:1 (1 m width for 3 or 4 meter length) (Mitsch and Gosselink, 2007; Kadlec and Knight, 1996; USEPA, 1999). An aspect ratio of 3:1 was set for this study. Thus, for the set surface area of 17.21 m^2, the width of the system was 2.4 m and the length was 7.2 m. The flow that can pass through the bed in subsurface conditions was determined using Darcy's equation. If the Darcy's Q was less that the design flow, then surface flow was possible. Thus the dimensions must be adjusted until Darcy's Q was equal to or greater than the design flow. The hydraulic conductivity of the PKS was assumed to be equal to that of gravel of the same size fractions, which was estimated as $480m^3/m^2/d$ (USEPA, 1999). The design cross-sectional area of the bed was 1.2 m^2. A slope of 1% was chosen for ease of construction (s = 0.01). The calculated Q was 5.76 m^3/d which was higher than the peak wet season flow of $1.28m^3/d$. Therefore surface flow was ruled out. The design drawings are shown in Appendix C.

3.5.2 Construction of the On-site Bio-remediation System

Septic tanks are usually made of reinforced concrete or concrete slab floor with block walls rendered watertight with neat cement. For the purpose of this study, two commercially available cylindrical plastic tanks were installed according to the two-chamber septic tank principle to reduce construction costs. The volume of each tank was 2.5 m^3. Sanitary tee pipe fitting were fixed at the inlet to direct the influent downward to provide more quiescent settling conditions and also at the outlet to prevent floating scum from exiting and clogging the wetland. Figure 3.7 shows the septic tanks after installation and the first chamber after 2 months of operation.

(**a**) **(b)**

Figure 3.7 (a) The two chamber septic tank system after installation; (b) First septic tank compartment after 2 months of operation.

The field-scale HSSF CW was constructed by excavating to the required depth, followed by leveling, compaction and rendering watertight with cement. Thereafter, it was lined with a high density polyethylene sheet (a waterproof material). The treatment area was filled and compacted with PKS of >5 mm size. Gravel of diameter between 10 - 20mm were used at the inlet and outlet areas in order to reduce clogging as recommended by UN-Habitat (2008). The inlet and outlet pipes were built to allow equal wastewater distribution and collection along the width of the wetland. A flow control pipe was installed such that a constant water level was maintained in the system. In May 2017, healthy young *Thalia geniculata* and *Colocasia esculenta* shoots were harvested from natural wetlands, washed to remove soil particles and separated into individual shoots. They were planted directly in the wetland and allowed to establish in stream water for two months with intermittent feeding of wastewater for nutrients, after which the system was connected to the septic tank. Three sampling ports were provided at 1m, 3m and 6m (9 in total) from the inlet distribution pipe to monitor tracer dynamics in the system. Transect at 1m, 3m and 6m from inlet were respectively named sampling paths 1,2,3; paths 4,5,6 and paths 7,8,9 as shown in the design drawing (Appendix C). The outlet pipe was the tenth sampling point. The sampling ports were made of perforated PVC pipes of 6cm diameter. Figure 3.8 shows the field-scale HSSF CW during planting and after 3 months of operation respectively. The principal construction steps are shown in Appendix D.

Figure 3.8 The HSSF CW during macrophyte planting and after 3 months of operation

3.5.3 Data Collection and Analysis

The HSSF CW was connected to the septic tanks and monitored from July 2017 to December 2017. To evaluate the treatment performance of the HSSF CW, field measurements, as well as influent and effluent samples were collected once a week. Samples were collected as previously described and immediately transported to the laboratory for physicochemical analysis. The removal efficiencies of the PKS based experimental field-scale HSSF CW was presented in terms of concentration removal efficiency (%) given as:

$$\text{Pollutant concentration reduction efficiency (\%)} = \frac{C_i - C_e}{C_i} \times 100 \qquad 3.8$$

3.6 Field and Laboratory Test methods

Field measurements were carried out for pH, EC, Temperature, TDS, NO_3-N, NO_2-N, PO_4^{3-} and aggregate thickness. pH was measured using RoHS pH-107 digital pH meter. The digital pH meter has a range of 0.0 to 14.0 and a resolution of 0.1. The pH was measured by immersing the sensing electrode into the wastewater sample and waiting until the readings on the display stabilized as shown in Figure 3.9(b). Buffer solutions of pH 4.01 and 6.86 were used to periodically calibrate the meter. This was necessary because the characteristics of the pH electrode changes with aging, and without proper calibration results will be off by at least several tenths of the unit.

(a) **(b)** **(c)**

Figure 3.9 (a) Field measurements using the digital meters; (b) pH measurement, (c) TDS/EC measurement

The EC, temperature and TDS were measured with Teika K12 digital handheld TDS-EC meter. The digital meter had a range of 0-9990 ppm for TDS, 0-9990 µs/cm for EC and 0.1-80 ^{o}C or 32.0-176.0 ^{o}F. These three parameters were measured by immersing the sensing electrode into the wastewater sample as shown in Figure 3.9(c). After a numerically stable display, the hold button was pressed and the meter taken out of the sample for reading. The shift button allowed for changing from TDS to EC measurement.

Nitrate nitrogen (NO_3-N), Nitrite nitrogen (NO_2-N) and Orthophosphate (PO_4^{3-}) were analyzed using the USEPA approved general purpose field test kits by Hach Company, USA (USEPA, 2016). For NO_3-N and NO_2-N analysis, the Nitrate Nitrite test kit model NI-12 (cat. No. 14081-00) with NitraVer 5 Nitrate Reagent Powder Pillow and NitriVer 3 Nitrite Reagent Powder Pillow for 5 ml sample was used. The kit had a range of 0 - 50mg/l for nitrate and 0 - 0.5mg/l for nitrite and sensitivities of 1mg/l and 0.01mg/l respectively. For PO_4^{3-} analysis, the Phosphate test kit model PO-23 (cat No. 224902) with PhosVer 3 Phosphate Reagent Powder Pillow for 5ml sample was used. The kit had a low range of 0.1 - 4mg/l and a high range of 1 - 40mg/l and sensitivities of 0.1mg/l and 1mg/l respectively. The test kits were purchased from Hach USA.

To analyze for NO_3-N and NO_2-N, the color viewing tube was rinsed severally with the wastewater and filled to the 5 ml mark after which the powder pillows were added. For PO_4^{3-} analysis 0.5 ml of the wastewater was discharged into the tube and diluted to the 5 ml mark with water after which the powder pillow is added. The tube was then closed with the stopper and shaken for one minute and left undisturbed for another minute for complete color development. The prepared tube was then placed in the right top opening of the color comparator. The second viewing tube was filled with untreated wastewater sample for NO_3-N and NO_2-N analysis and demineralized water for PO_4^{3-}. It was then placed in the color comparator, the color disc in the comparator rotated to obtain a color match, which gave the sample concentrations. Samples containing more the 50 mg/l of Nitrate, 0.50 mg/l of Nitrite or 40mg/l of PO_4^{3-} were measured by diluting the sample before measurement and multiplying the result by the dilution factor.

Standard laboratory methods, as described in APHA (1998), were utilizeds to examine the physicochemical parameter of the effluent samples for BOD, COD, TSS and NH_4-N as reported here: For BOD, method 5210B (5-Day BOD Test) which involved filling a sealed bottle with diluted and seeded sample, and incubating for five days was utilized. Oxygen uptake during incubation is used for BOD measurement. For COD, method 5220B (Open Reflux method) was utilized. For TSS method 2540 D (TSS dried 103-105°C) at was used. This involved filtering well-mixed sample through a weighed filter paper. The residue retained on the filter paper is dried to a constant weight at 103 to 105°C. The increase in weight of the filter papaer represented the TSS. For NH_4-N, method 4500F (Phenate method) was utilized.

The variation in the shell thickness of the PKS aggregate over the 20 months period was measures using a Vernier Caliper. The procedure involved selecting representative shell samples of equal thickness. The samples were tied in a wire mesh and buried in the pilot wetland. A wire was attached to the wire mesh, extending to the surface for easy extraction of the sample. After extraction the thickness of the samples were measured. Specific gravity was determined following the ASTM C127-07 (2007) procedure. It was determined by washing the aggregate sample thoroughly to remove fines and immersing it in water for 24 hours using the wire basket. The basket and sample was then weighed while suspended in water. Then the sample was transferred to an absorbent clothe, surface dried and weighed. Also the empty wire basket was weighed while suspended in water. The specific gravity was computed as:

$$\text{Specific Gravity} = \frac{W_3}{W_3-(W_1-W_2)} \qquad 3.9$$

Where W_1 is the weight of saturated aggregate suspended in water (g); W_2 is the weight of wire basket suspended in water (g); W_3 is the weight of saturated surface-dried aggregate in air (g)

The aggregate crushing value of the PKS substrate was determined following the BS 812 (1990) procedure by oven drying aggregates passing through the 12.5 mm and retained on the 10mm sieve at a temperature of 100 ^{o}C to 110 ^{o}C for 3 to 4 hours. A cylindrical measure was then filled with the aggregate and tampered with 25 strokes. The weight of the aggregate was the measured (weight A). The surface of the aggregate was leveled and the plunger inserted. It was then placed in a compression testing machine and loaded at a uniform rate to achieve 40 tons, after which the load was released. The sample was sieved through a 2.36 mm sieve and fraction passing through the sieve was weighed (weight B). The aggregate crushing value was computed as:

$$\text{Aggregate Crushing Value} = \frac{\text{Weight B}}{\text{Weight A}} \text{ x } 100\% \qquad 3.10$$

3.7 Statistical Methods

Statistical analyses were performed using SPSS Version 21. For wastewater characterization at the slaughterhouses, the means and standard deviation of the raw data were calculated. The mean values of the measured wastewater physicochemical parameters from the Agulu slaughterhouses, being the project location, were compared to the means from the other slaughterhouses for differences using one-way multivariate analysis, with statistical significance set at $p \leq 0.05$. Tukey post hoc test was used for multiple comparisons. Correlation analysis was used to measure the strength of statistical relationship between physicochemical parameters of the Agulu slaughterhouse wastewater.

For the pilot study, the differences of mass removal rates amongst the wetlands were analysed with the one-way ANOVA. Statistical replication for the study was carried out with sequential experiments conducted on the same systems. The Shapiro–Wilk test was used to examine the data for normality before the ANOVA. If the data was normally distributed, no transformation was done, but if data was not normally distributed, the data were transformed either using square root or log transformation. 95% confidence interval was set for statistical significance for the differences in the means of the different treatments evaluated, and thus differences were considered significant at $p \leq 0.05$. For multiple post hoc tests, Tukey procedure was carried out. Sample analysis is presented in Appendix F. The difference between the means of the influent and effluent parameter concentrations of the PKS based experimental field-scale HSSF CW were also analysed using one-way ANOVA.

3.8 Tracer Experiment

In January 2018, the HSSF CW was disconnected from the septic tank and loaded with 5 m^3 of tap water daily for 8 days to ensure low background concentrations for tracer test, which was conducted from 12th - 25th January 2018. During the tracer study, clean borehole water was used and an inflow of 0.75 m^3/d was maintained throughout the experiment based on the design average daily wastewater inflow to the CW. This was necessary so as to ensure controlled and constant inflow rates. Common salt (NaCl) was used as the tracer material. Determining the amount of tracer to be added for a pulse input test can be difficult (Axelsson et al., 2005). To achieve a target peak concentration of approximately 20 times the background concentration, the dosing calculation proposed by Teefy (1996) was used as given in Equation 3.11.

$$Mass\ of\ tracer\ needed\ = V\ *\ 20\ \ *\ C_{background}\ \ *\ dosing\ factor \qquad 3.11$$

Where V is the volumn of basin to be teseted (L); $C_{background}$ is the background concentration (mg/l); Dosing factor recommended based on the expected hydraulic efficiency of the sysem is as follows: 1 for poor hydraulic efficiency, 0.6 for average hydraulic efficiency and 0.2 for superior hydraulic efficiency

A tracer solution of 5g NaCl/l was prepared by adding 250g of NaCl to 50 liters of tap water and mixied to dissolve completely. To avoid temperature stratification after tracer injection, the borehole water was left onsite one day before the test. The solution was then injected into the wetland inlet. The primary method of data collection for the tracer experiment was conductivity measurement. Electrical conductivity (EC) routinely used in environmental fields provide a rapid, inexpensive and generally reliable proxy for the ionic content of a solution. Prior to the tracer experiment, it was necessary to determine the relationship between EC and the amount of salt tracer in a solution. This calibration was done by dissolving small amounts of salt in the water to be used for the experiment and measuring the corresponding electrical conductivity as shown in Appendix E. The obtained relationship is given as:

$$C_{NaCl}(mg/l) = 0.184\ *EC(\mu S/cm) - 126.1 \qquad 3.12$$

EC measurement was done before the salt tracer injection to measure the background noise. Few minutes after tracer injection, EC was measured at the 9 sampling ports and outlet. Subsequently sampling was done every day. Daily water loading and EC measurements were carried out until the EC value was equal to the background noise. The outflow was collected in buckets and measured to determine the volume.

3.8.1 Application of Residence Time Distribution Models

Two models are constantly used to analyze wastewater treatment systems in the literature which are plug flow with dispersion and tank-in-series models. The hydraulic characteristics of the PKS-based experimental field-scale HSSF wetland were investigated using these models.

Hydraulic parameter values that best reflected the system behaviour with respect to tracer transport were obtained using these models. Preliminary values of mean transit time of tracer and dispersion parameters were calculated for the outlet point by the method of moments. The computational procedure for the tracer RTD is shown in Appendix H. These estimates were used as starting values for the fitting procedure by which the solutions of the transport equations derived for the *TIS* and *PFD* models (Equations 3.14 and 3.15 respectively), were iteratively fitted to the experimental RTD.

The tracer concentration in the N^{th} tank is given as:

$$C(t) = \frac{N^N t^{N-1}}{\tau^N (N-1)!} exp[-\frac{Nt}{\tau}] \qquad 3.13$$

N is usually changed from a discrete integer to a non integer, due to the fact that most CWs function as a few number of tanks in series (Kadlec and Wallace, 2009). This is done by replacing the factorial function with the Γ function, which increases the flexibility of the data fitting process. Thus *TIS* model is represented as a Gamma distribution *f(t)* of detention time given as (Kadlec and Wallace, 2009):

$$f(t) = \frac{N^N t^{N-1}}{\tau \Gamma(N)} \exp[-\frac{Nt}{\tau}] \qquad 3.14$$

Gamma distribution function are available in Excel® software (GAMMADIS and GAMMALN) and it returns values of f for the time t and the parameters N and τ. The Gamma function $\Gamma(N)$ was calculated using the functions EXP(GAMMALN(N)) in Excel®.

Analytical solution of the one-dimensional *PFD* modeld, in case of instantaneous injection and detection in fluid flux is given as (Levenspiel and Turner,1972; Kreft and Zuber, 1978):

$$C(t) = \frac{A}{Q\tau\sqrt{4\pi(D/Ux)(t/\tau)^3}} \exp\left[-\frac{(1-t/\tau)^2}{4(D/Ux)(t/\tau)}\right] \qquad 3.15$$

Where C is the tracer concentration (mg/l); N is the number of tanks in series; t is the time from injection (days); τ is the mean residence time (days), A is the amount of tracer (mg); Q is the discharge (m^3/day), D is the dispersion coefficient, U is the rnean water velocity (m/day), x is the distance from injection point (m). D/Ux = dispersion number (P_D)

Two parameters, N and τ were simultaneously adjusted in the *TIS* model to obtain the values which minimized the SSQE between the measured and the predicted residence time distribution. For the PFD model, P_D=D/Ux and τ were simultaneously adjusted.

3.9 Computational Fluid Dynamics Modeling of the Field-Scale HSSF-CW Hydrodynamics

A two dimensional (2D) CFD model was developed to simulate the hydrodynamic behaviour of the experimental field-scale HSSF CW, using the version 5.3a of the finite element software COMSOL Multiphysics®. The software is a powerful tool that allows the modelling of different types of physical phenomena. Flows can be simulated in various forms like stationary or time-dependent, under laminar or turbulent conditions and models can also be built in two or three dimensional spaces. Different complex geometries and physical properties can be represented using the software and the graphical user interface is very flexible and contains all the tool needed to build a successful model. The modeling process in COMSOL consists of six main steps: Selection of appropriate physics mode; Setting up the sub-domain equations and boundary conditions in the physics mode; Drawing or importing the model geometry; Meshing the model geometry; Solving in the solve mode and Post-processing. All the steps are available in the graphical user interface of the software.

The *laminar flow* and *transport of diluted species in porous media* interfaces of COMSOL Multiphysics were used to build the model. They were used to simulate tracer transport in the system. The laminar flow interface was chosen because the flow conditions of the experimental HSSF-CW were well within the limits of the laminar flow regime.

3.9.1 Governing Equations

The Navier-Stokes equations given below are the basis of Laminar Flow of COMSOL and are given as (Comsol, 2017):

$$\frac{\rho}{\partial t} + \nabla.(\rho u) = 0 \qquad 3.16$$

$$\rho\frac{\partial u}{\partial t} + \rho(u.\nabla)u = \nabla.[-pI + \tau] + F \qquad 3.17$$

Where

ρ is density; u is velocity; *p* is pressure; τ is viscous stress tensor and F is volume force vector.

Equation 3.16 is conservation of mass and Equation 3.17 is conservation of momentum. For a Newtonian fluid, which has a linear relationship between stress and strain, the viscous stress tensor is given as:

$$\tau = 2\mu S - \frac{2}{3}\mu(\nabla.u)I \qquad 3.18$$

Where μ is the dynamic viscosity (Pa.s) and S is the strain-rate tensor given as:

$$S = \frac{1}{2}(\nabla u + (\nabla u)^T) \qquad 3.19$$

Thus, for a compressible flow the momentum equation becomes:

$$\rho \frac{\partial u}{\partial t} + \rho u \,.\, \nabla u = -\nabla p + \nabla \,.\left(\mu(\nabla u + (\nabla u)^T) - \frac{2}{3}\mu(\nabla \,.\, u)I\right) + F \qquad 3.20$$

When the temperature variations in the flow are small, a single-phase fluid can often be assumed incompressible; that is, ρ is constant or nearly constant. For constant ρ, the continuity equation (Equation 3.16) reduces to

$$\rho \nabla.(u) = 0 \qquad 3.21$$

and Equation 3.20 becomes

$$\rho \frac{\partial u}{\partial t} + \rho(u \,.\, \nabla)u = \nabla \,.\, [-pI + \mu(\nabla u + (\nabla u)^T)] + F \qquad 3.22$$

Transport of diluted species interface was used for the tracer studies. The phenomena of tracer diffusion and convection are modeled by the mass conservation equations given as:

$$P_{1,i}\frac{\partial C_i}{\partial t} + P_{2,i} + \nabla \,.\, \Gamma_i + u \,.\, \nabla c_i = R_i + S_I \qquad 3.23$$

$$P_{1.i} = \varepsilon_p \qquad 3.24$$

$$P_{2.i} = c_i \frac{\partial \varepsilon_p}{\partial t} \qquad 3.25$$

$$N_i = \Gamma_i + uc_i = -D_{e,i}\nabla c_i + uc_i \qquad 3.26$$

$$D_{e,i} = \frac{\varepsilon_p}{\tau_{F,i}} D_{F,i} \qquad 3.27$$

where

c_i is concentration; Γ_i is diffusive flux; R_i rate of reaction; S_i is source; $D_{e,i}$ is effective diffusion, $D_{F,i}$ is molecular diffusion and $\tau_{F,i}$ is tortuosity.

If porosity and diffusion are constant, $P_{2,i} = 0$ and $\nabla \,.\, \Gamma_i = \nabla \,.\, (-\varepsilon_p D_{F,i} \nabla c_i) = -\varepsilon_p D_{F,i} \nabla^2 c_i$. Also under such conditions Ri = 0, Si = 0 and $\tau_{F,i} = 1$, the system of Equations 3.23 - 3.27 reduce to the convection-diffusion equation given as:

$$\varepsilon_p \frac{\partial c}{\partial t} + u \,.\, \nabla c = \varepsilon_p D \nabla^2 c \qquad 3.28$$

3.9.2 Initial and Boundary Conditions

Apart from the domain equations, proper boundary conditions were selected. Normal inflow velocity, $(u = -\, u_o\, n)$ were specified for the fluid flow at the inlet and outlet, and pressure was set at as p = 0. At the walls, no slip boundary conditions were set. For tracer transport in the porous media, the initial value of concentration inside the constructed wetland was chosen to be zero (since the salt concentration in the borehole water used for the study was negligible). The concentration of tracer at the inlet was specified as a time-dependent inflow value of (85.5*rect1(t[1/s]) mol/m^3. At the outlet, it was specified that the mass flow through the boundary was convective dominated (-n . $D_i \nabla c_i$ = 0). This assumes that any mass flux due to diffusion across this boundary is zero. An insulation boundary condition was specified at the boundaries, thus no mass is transported across the boundaries. Density and dynamic viscosity of water were set at 1000 kg/m^3 and 0.001 Pa.s respectively (https://ascelibrary.org). Average velocity of flow into the constructed wetland (9 x 10^{-6} m/s) was determined by dividing the volumetric flow rate by the cross sectional area, while the diffusivity of NaCl in water was set at 1.607 x 10^{-9} m/s (Essays, 2018).

3.9.3 Geometry and Mesh Generation for the CFD Model

A 2D geometry of the experimental field-scale HSSF-CW (8.2 m length by 2.4 m width) was built in the Geometry mode of COMSOL. The choice of 2D model in this research was because of non availability of a high performance computing facility needed to carry out 3D modelling. The macrophyte shoot positions were measured in the experimental HSSF-CW and vegetation was modeled as cylinders with uniform diameter (0.06 m representing the stems), as suggested by Kadlec (1990). The physics controlled mesh sequence type was selected, so as to allow COMSOL select the appropriate mesh type for the specified flow condition, and the size of the element was selected to be normal. The mesh partitions the geometric model into small units of simple shape. The geometry of the HSSF-CW and generated mesh is shown in Figure 3.10.

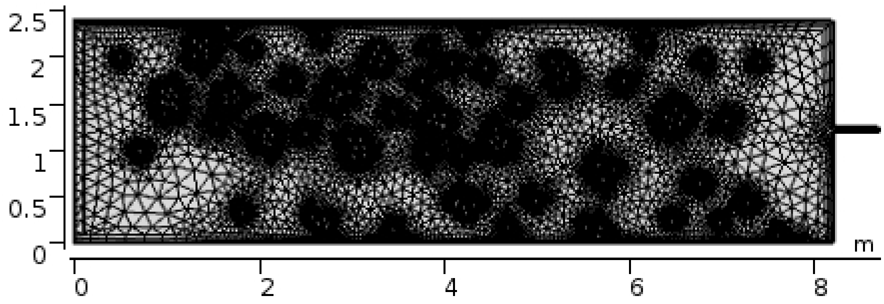


Figure 3.10 Rectangular geometry and physics-controlled mesh for the HSSF CW

3.9.4 Solving the CFD Model

Figure 3.11 shows the graphical user interface (GUI) of COMSOL Multiphysics Version 5.3a. A typical representation of the wetland is shown in the geometry mode. Each tool on the GUI does a specific function that is specified by the user. After the mesh parameter has been set up, the model is ready to be run for any solution that has been specified. The first step of the simulation was the steady state determination of hydrodynamic components of flow, particularly velocity field (u) and pressure (p), using the laminar flow interface. Determination of these components for the flow were necessary for the next step. The second step was the utilization of transport of diluted species interface to simulate the time-dependent tracer concentrations (c) in the system in

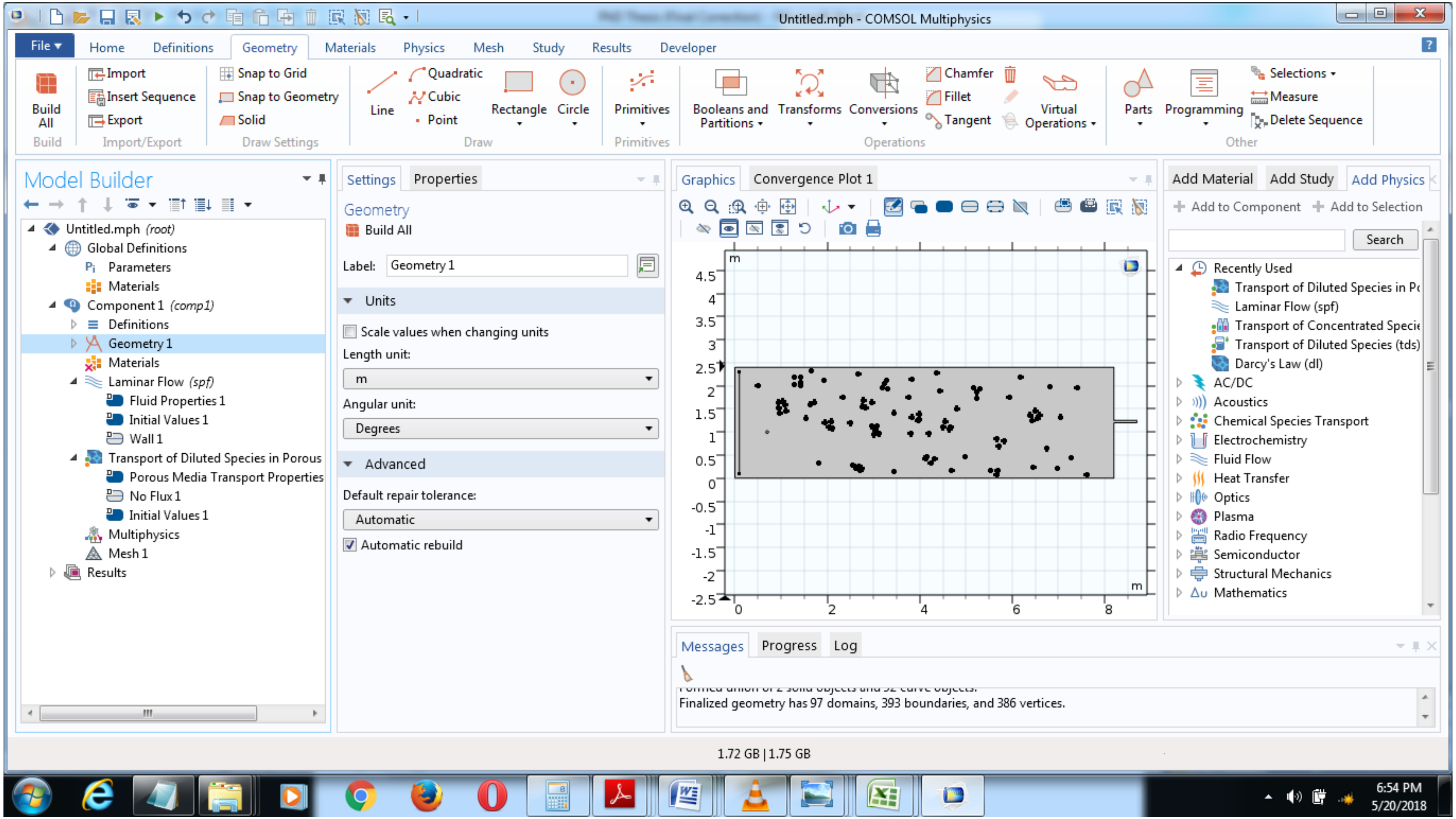


Figure 3.11 Graphical user interface (GUI) of COMSOL Multiphysics Version 5.3a

order to determine tracer residence time, and the tracer was monitored with two hour resolution at the outlet of CW. The simulations were performed on a laptop computer (Intel ® Core(TM) i5-2430M CPU @ 2.40GHz with 8.00GB RAM). The solution converged and produced a residence time distribution curve in the CFD simulation as would be presented in results chapter. This demonstrated that the developed model of the HSSF CW hydrodynamics was correct and can be used in further CFD model simulations.

3.10 Simulation of the Effect of Uniformly Distributed Vegetation and Baffles

Well distributed vegetation has been reported to force the hydraulic behaviour of HSSF CWs towards plug flow. The effect of uniformly distributed shoots; with a density of 25shoots/m^2 on the residence time water in HSSF CW was evaluated. However, uniformly distributed vegetation may be difficult to achieve in reality, thus calling for other physical design intervention to improve the hydraulic efficiency of HSSF CWs. The use of baffles in wastewater treatment systems has been reported by a number of previous studies (Su *et al*., 2009; Tee *et al*., 2012). It is with this knowledge that baffles were introduced to the HSSF CW model and the effect of such introduction was verified. Figure 3.12 shows the geometry of the uniformly distributed vegetation and the baffled wetlands.

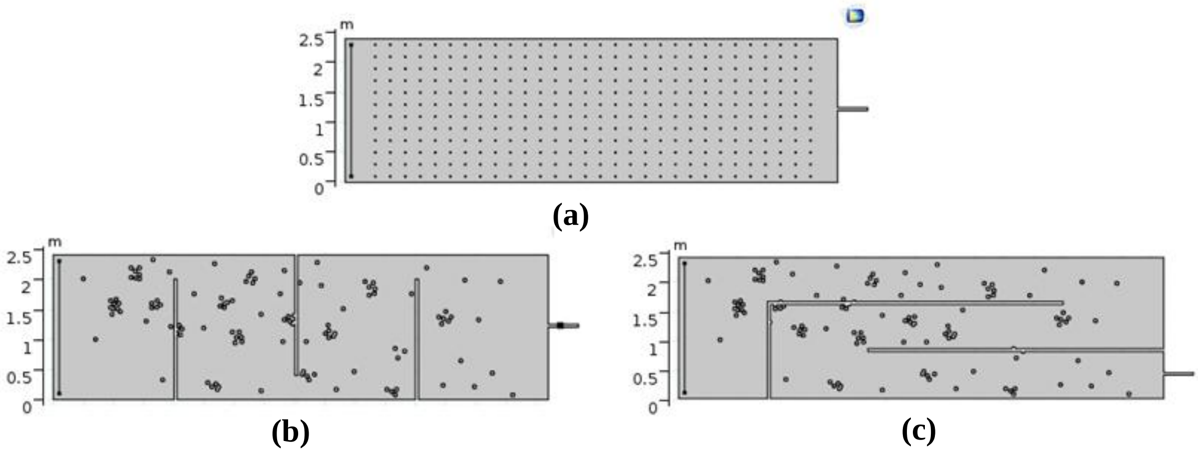
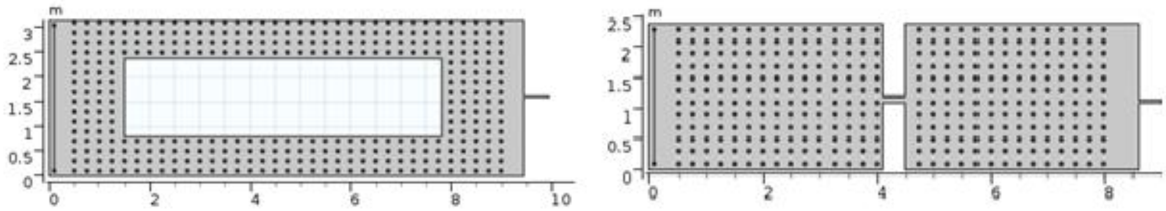


Figure 3.12 Modelled geometries: (a) uniformly distributed vegetation; (b) vertical baffles; (c) Horizontal baffles

3.11 Simulation of RTD under different Design Configurations

Wetland configurations have been reported to influence the extent of hydraulic short-circuiting and stagnation in many CWs (Zounemat-Kermani *et al*., 2015). Therefore evaluation of the effect of different configurations on the residence time distribution, using CFD modelling is necessary. Two aspect ratios of a rectangular wetland (1:2 and 1:4) were modelled and the redidence time distribution simulated and compared to the residence time distribution of the full-scale HSSF CW model with aspect ratio of 1:3. The CFD model was also extended to determine the residence time distributions under two alternative HSSF CW design configurations (Rectangular basin with island and a two cell system). These alternative configurations were also compared to the full-scale HSSF CW model. All ponds had surface areas of approximately 17 m^2, and the alternative configurations had an aspect ratio of 1:3. Figure 3.13 shows the geometries of the alternative designs.



(a) (b)

Figure 3.13 Modelled geometries: (a) Rectangular basin with island (b) two cells

3.12 Multiple Regression Analysis

Multiple regression analysis (MRA) has been found to provide useful and accurate models for simplified description and analysis of CWs performance (Babatunde *et al.*, 2011; Tomenko *et al*., 2007). The fit of statistical models developed from MRA was evaluated in this study. To investigate if any relationship exists between the wastewater quality parameters, correlation analysis was conducted prior to the MRA. This was necessary to determine parameters necessary for the analysis. The analysis analysis was carried out for each of the dependant variables (BOD, TSS, NH_4-N, NO_3-N and PO_4^{3-}) using a combination of one to three predictor variables. This was done to develope the optimum model from the least combination of inputs variables so as to minimize errors. A confidence interval of 95% was set for the analysis and coefficient of determination (R^2), the significant F value, the p values and the standard error of the estimation (S) were used for model evaluation.

3.13 Structured Interview

A survey by structured interview method was conducted to assess the opinion of slaughterhouse operators and butchers about the technology. People that witnessed the construction and operation of the HSSF CW were interviewed. The survey was conducted towards the end of the study. The aim of the survey was to determine their previous knowledge about the application of CWs for general wastewater treatment, their opinion on the advantages and disadvantages of the system, based on their observations; and finally, if they are likely to recommend the technology to others.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Slaughterhouse Wastewater Characterization Results

The results of the studies on available facilities, the production and management of wastewater and the strength of the wastewater from the seven slaughterhouses studied are presented in this section.

4.1.1 Facilities, Wastewater Production and Management at the Slaughterhouses

The Eke-Awka Etiti, Agulu and Amansea slaughterhouses were exclusively for beef, while the Umunya, Nkwo-Nnewi, Eke-Ekwulobia and Ochanja were mixed slaughterhouses for cattle and goats. The typical meat processing operations in most slaughterhouses include slaughtering, bleeding, skinning, evisceration and splitting into different sizes for transport to the markets. After the animal is stuck with a knife, the blood flows from the animal to the slaughter slab. A blood pit for blood collection was provided at the Agulu slaughterhouse. The blood is dried and used for the production of animal feeds. At the six other slaughterhouses there were no provisions for blood collection, the blood drains into the effluent channel and mixes with the wastewater.

Slaughterhouses have been classified, based on the number of animals slaughtered as: large scale - when more than 200 cattle per day are slaughtered; medium scale - when slaughtering between 50 and 200 cattle per day; and small scale - when fewer than 50 cattle per day are slaughtered. According to Chukwuma *et al*. (2016), the average daily slaughter rate for cattle in slaughterhouses was estimated at: 8.5 in the Eke-Ekwulobia slaughterhouse; 8 in the Agulu slaughterhouse; 23 in the Amansea slaughterhouse; 35 in the slaughterhouse at Eke-Awka Etiti; 10.5 at the Nkwo-Nnewi slaughterhouse; 65 in the Umunya slaughterhouse and 70 in the Ochanja slaughterhouse. Thus, the Umunya and Ochanja slaughterhouses can be classified as medium-sized, while the Eke-Ekwulobia, Agulu, Amansea, Eke-Awka Etiti and Nkwo-Nnewi slaughterhouses can be classified as small-scale. The result of a general comparison of the facilities in the slaughterhouses to what is expected of a standard slaughterhouse is shown in Table 4.1. The facilities have been assessed for presence and functionality.

Table 4.1 Comparison of facilities at the seven slaughterhouses to a standard Abattoir

	Slaughterhouse						
Facilities	Umunya	Nnewi	Amansea	Ekwulobia	Agulu	Awka Etiti	Ochanja
Lairage	2	2	2	2	2	2	2
Slaughter slab	1	1	1	1	1	1	1
Cold room	-	-	-	-	-	-	2
Drainage system	-	2	-	2	2	1	2
Water supply	1	2	2	2	2	2	1
Electricity supply	1	1	-	-	-	1	1

Key: (-) = absent; 1 = good; 2 = poor

According to Fonseca (2000), very clean water that meets the standard for drinking should be used for cleaning purposes. There was no portable water supply in most slaughterhouses. Apart from the Umunya and Ochanja slaughterhouses, where boreholes were provided, the other slaughterhouses use water from unidentified sources, which were mostly obtained from suppliers and stored in plastic tanks. Occasionally the butchers extracted water from the nearby stream if there were delays in the supply. Use of water of questionable quality leads to the contamination of the carcass as stated by Bello et al. (2011) in their observation of increase in *E. coli* counts in beef carcasses in some Nigerian slaughterhouses as a result of the use of non-portable water. Also none of the slaughterhouses use hot water for cleaning, which is considered very important due to the oily nature of their products. When detergent is added to hot water, it facilitates the proper cleaning of slaughterhouse floors and equipment.

Electricity, which is necessary for various operations, such as refrigeration and lighting, was lacking in some facilities. According to Akinro *et al*., (2009), the absence of electricity in slaughterhouses leads to incorrect processing operations. Given the important role of lairage for animal restraint and ante-mortem inspection, the findings of this work showed that the lairage were in most cases makeshift structures built with bamboo. Therefore, no ante-mortem examinations were performed in slaughterhouses, which were similar to report of Bello *et al*., (2008) as regards the lack of ante-mortem inspection in most slaughterhouses in Northern Nigeria. Other facilities such as cold room, rail system, veterinary laboratory, disinfection and first aid room were missing in all slaughterhouses.

The estimated average daily wastewater generation at the seven slaughterhouses from October 2016 to December 2016, assuming 80% of the water input is discharged as wastewater (John, 1995), is shown in Table 4.2.

Table 4.2 Estimated mean wastewater production at the slaughterhouses

Slaughterhouse	Water Input (m^3/d)	Wastewater Production (m^3/d)
Umunya	4.20	3.36
Nkwo-Nnewi	1.30	1.04
Amansea	2.30	1.84
Eke-Ekwulobia	0.60	0.48
Agulu	0.20	0.16
Eke-Awka Etiti	2.50	2.0
Ochanja	5.60	4.48

Wastewater production was highest in the Ochanja slaughterhouse, with a production of 4.48m^3/d, and a wastewater production of 0.16m^3/d from the Agulu slaughterhouse was lowest. The variation in wastewater production in the slaughterhouses was expected because the wastewater production is influenced by number of animals that are processed in a slaughterhouse. The average volume of water per head in the seven slaughterhouses was estimated at: 0.06m^3 at the Eke-Ekwulobia slaughterhouse; 0.02m^3 in the Agulu slaughterhouse; 0.08m^3 in the Amansea slaughterhouse; 0.06 m^3 in the Eke-Awka Etiti slaughterhouse; 0.1m^3 in the Nkwo-Nnewi slaughterhouse; 0.05m^3 in the Umunya slaughterhouse and 0.06m^3 in the Ochanja slaughterhouse. The average volume per head of the seven slaughterhouses was thus estimated at 0.061 m^3. This is much lower than the benchmark of 0.7 to 1.0m^3 per cow set by UK Environment Agency (Environment Agency, 2008).

The low water consumption can be attributed to the lack of portable water supply in most slaughterhouses. There was serious water rationing in the facilities, which reportedly translates into an increased concentration of pollutants (Masse and Masse, 2005). The low volume per head can also be due to the fact that no further processing is carried out in the slaughterhouses, except slaughtering, eviscreation and splitting into manageable sizes for easy transport to the different markets where further cutting is carried out. Other processes known to contribute to water consumption, such as spraying cattle with water for evaporative cooling to prevent hyperthermia, pre-eviscreation washing, carcass washing, etc. were not carried out in the slaughterhouses.

According to Johns (1995), the recovery and subsequent management of waste in slaughterhouses can be reasonably efficient. The same can not be said of most slaughterhouses in the state of Anambra. The management of solid waste in the slaughterhouses studied was very poor; heaps of paunch manure was common sights around the slaughterhouses. Of the seven slaughterhouses examined, it was only in the Agulu and Eke-Awka Etiti slaughterhouses that conscious efforts at wastewater collection and storage were made by the provision of septic tanks as presented in Figure 4.1.

(a) (b)

Figure 4.1 Septic tanks for wastewater collection (a) Eke-Awka Etiti slaughterhouse (b) Agulu slaughterhouse

At the time of the investigation, however, the septic tank in Agulu slaughterhouse was full and had yet to be evacuated, which resulted in the outflow of effluent into the environment. In the other slaughterhouses no provisions were made for the collection, storage or treatment of wastewater. The effluents were flushed into drains that emptied into shallow depressions in the slaughterhouse premises or in public drainage as shown in Figure 4.2.

Figure 4.2 Effluent discharges into the environment at the slaughterhouses

4.1.2 Physicochemical Properties of Wastewater at the Slaughterhouses

4.1.2.1 pH, Temperature and Electrical Conductivity

The variations of the mean pH at the seven slaughterhouses are shown in Figure 4.3.

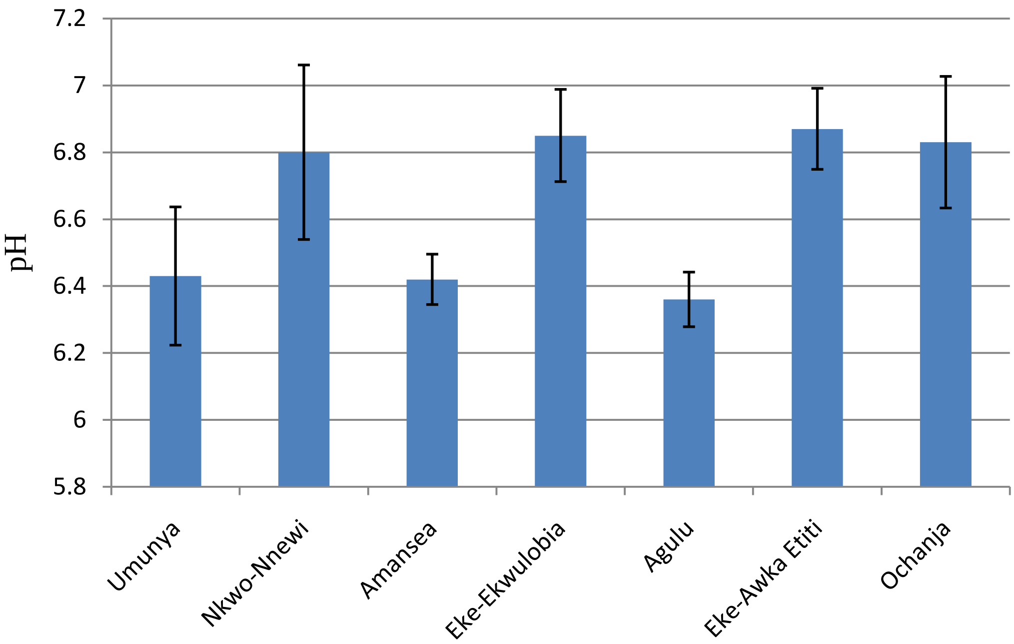


Figure 4.3 Variation of mean pH at the seven slaughterhouses, vertical bars are standard errors of the mean of n = 6.

The average pH varied from 6.4 ± 0.07 at the Amansea slaughterhouse to 6.9 ± 0.14 at the Eke-Ekwulobia slaughterhouse. The pH varied from slightly acidic to basic, with the average pH for slaughterhouses in the state of Anambra, based on this study, estimated as slightly acidic with a value of 6.7. The pH level in the wastewater from the slaughterhouses was within the FEPA (1991)/WHO (2004) set limits of 6.0-9.0 for effluent discharge into water bodies. The pH values obtained in the study were comparable to previous ranges of 6 - 10 reported by Mittal (2004) for slaughterhouses in Europe. A range of 7.6 - 8.2 was also reported by Akan *et al*., (2010) for slaughterhouse wastewater samples from the Maiduguri Metropolis, Nigeria.

The variations of the mean temperature at the seven slaughterhouses are shown in Figure 4.4.

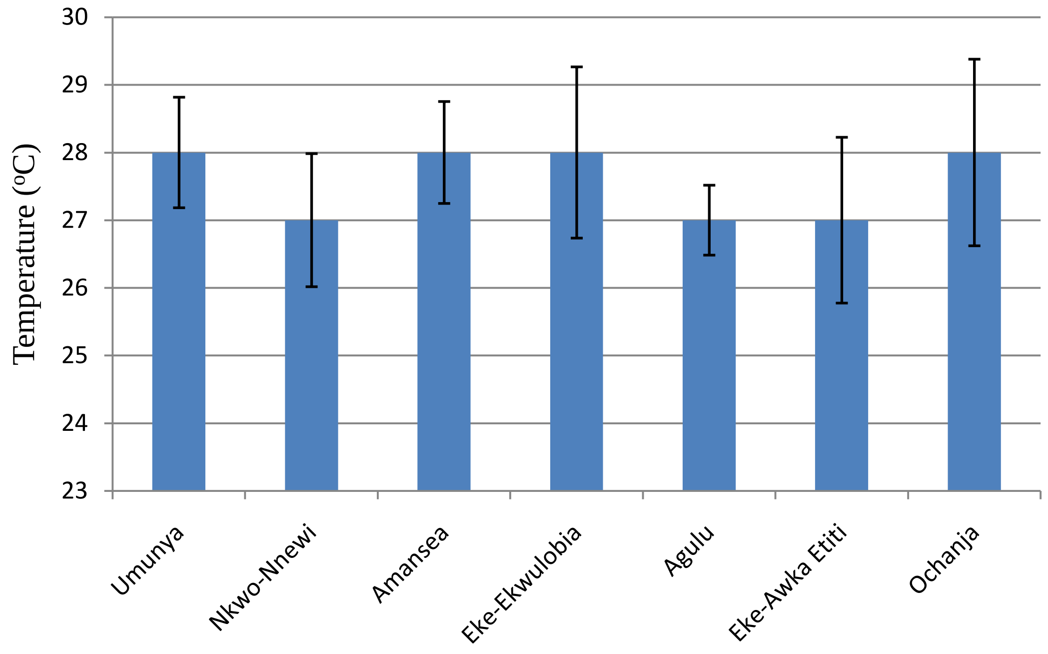


Figure 4.4 Variation of mean temperature at the seven slaughterhouses, vertical bars are standard errors of the mean of n = 6.

The temperature of the effluent from the slaughterhouses varied from 26 - 27°C in the rainy season (October 2016) to 29 - 30°C in the dry season (December 2016). The average temperature during the study period varied from 27.5°C in the Eke-Awka Etiti slaughterhouse to 28.5°C in the Ochanja slaughterhouse, as shown in Figure 4.6. The higher temperature values obtained during the dry season can be attributed to the high intensity of sunlight during the period that increased the ambient air temperature and also the effluent temperature. However, the temperature values recorded in all slaughterhouses fell within the FEPA (1991) recommended limit of <40°C. These ranges were in agreement with a similar study conducted in Nigeria, with a range of 26.6°C to 29.17°C (Ekanem *et al*., 2016).

The variations of the mean electrical conductivity at the seven slaughterhouses are shown in Figure 4.5.

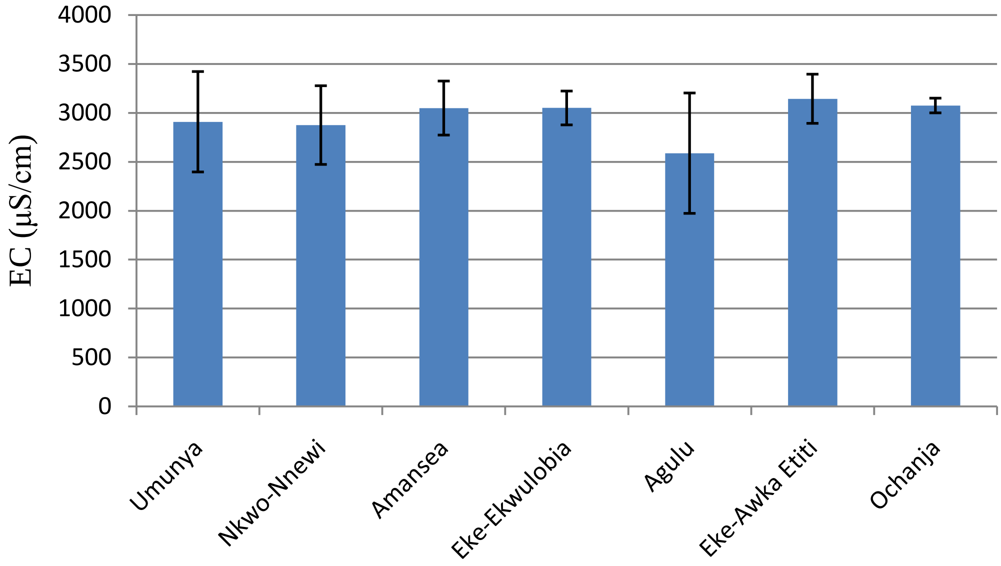


Figure 4.5 Variation of mean electrical conductivity at the seven slaughterhouses, vertical bars are standard errors of the mean of n = 6.

The electrical conductivity (EC) of water is an expression of its ability to conduct electric current. The average EC values for effluent from the slaughterhouse ranged from 2588.8uS/cm in the slaughterhouse of Agulu and 3144.8uS/cm in the Eke-Awka Etiti slaughterhouse (Figure 4.6). The EC values recorded in all slaughterhouses were much higher than the FEPA (1991) limit of 1000 µS/cm for wastewater discharge. The high conductivity values can be attributed to salts present in animal waste and intestinal contents, which is an indication of substantial dissolved ions.

4.1.2.5 Total Suspended Solids, Total Dissolved Solids and Total Solids

Figure 4.6 shows the variation of TSS, TDS and TS at the studied slaughterhouses. TSS mean values varied from 460.7mg/l at the Agulu slaughterhouse to 885.8mg/l at the Ochanja slaughterhouse. The mean TSS levels at the slaughterhouses exceeded the set limit of 30mg/l by FEPA (1991). TDS mean values for the effluent from the slaughterhouses, which were within the 2000 mg/l set limit of FEPA (1991), varied from 1416.0mg/l at the Umunya slaughterhouse and 1667.8mg/l at the Ochanja slaughterhouse. Average values of TS ranged from 2037.4mg/l at the Eke-Ekwulobia slaughterhouse and 2553.7mg/l at the Ochanja slaughterhouse. The values obtained from the study were found to be very variable when compared with similar studies. Ekanem *et al.,* (2016) reported higher values of 2114.27mg/l during the rainy season and 2507.93mg/l during the dry season, for TDS and 2690.67mg/l during the rainy season and 2133.33mg/l during the dry season for TSS. Range of values reported by Akan *et al.*, (2010) were 856.0 - 1080.0mg/l for TSS and 3200 - 3480mg/l for TDS. Values of 4688.00 - 11053mg/l for TDS and 6348 - 12145mg/l for TSS were reported by Ojo and Alamuoye (2015).

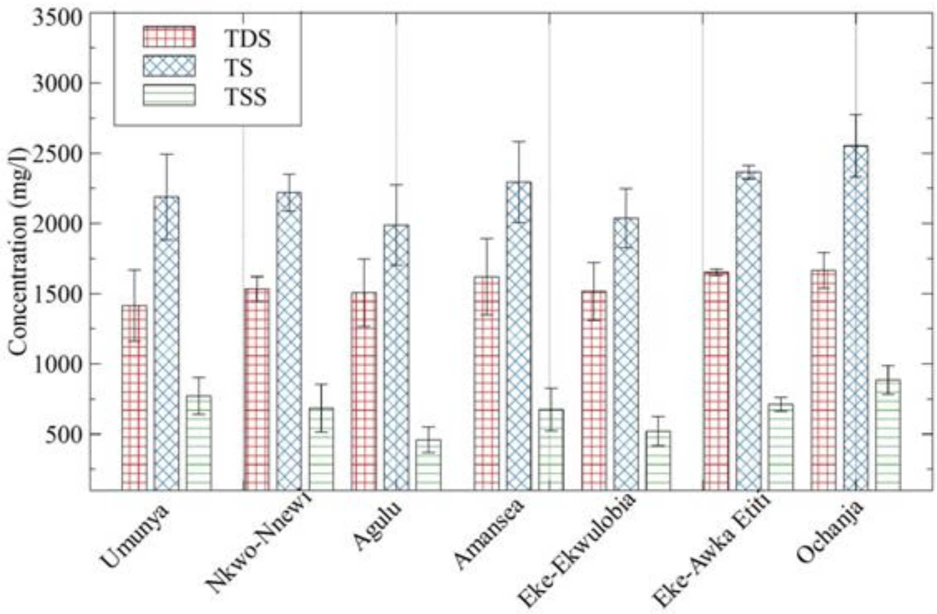


Figure 4.6 Variation of TSS, TDS and TS at the seven slaughterhouses, vertical bars are standard errors of the mean of n = 6.

However, Atuanya *et al.*, (2012) in their study of effluent quality of government and private abattoirs in Benin City, Nigeria, reported very low TSS values of 59mg/l and 6lmg/l for government and private abattoirs respectively. Presence of solids in abattoir wastewater can be attributed to solid by-products such as paunch manure, blood, fats and soft tissues that are removed during cutting and this result to an aesthetically unpleasant odour.

4.1.2.6 Biochemical Oxygen Demand and Chemical Oxygen Demand

The mean BOD and COD values obtained from the study were found to be very high. As shown in Figure 4.7, BOD concentrations varied from 613.7mg/l at the Agulu slaughterhouse to 1049.3mg/l at the Ochanja slaughterhouse. COD mean concentrations ranged from 1020.7mg/l to 1698.9mg/l at the Agulu and Ochanja slaughterhouses respectively. The values were found to be much higher than FEPA (1991) limits of 50mg/l (BOD) and 80mg/l (COD) for the discharge of effluent into surface water.

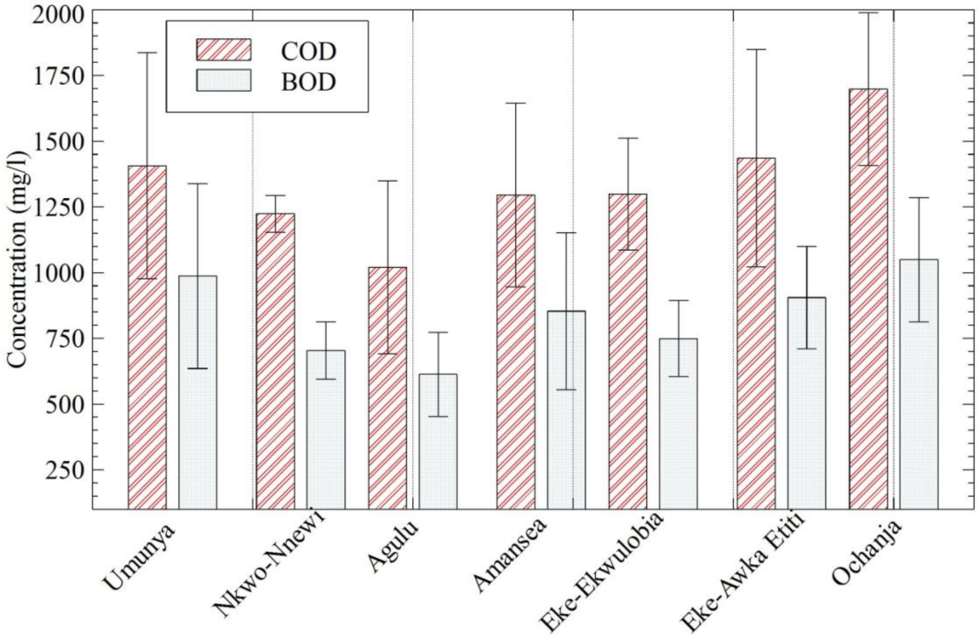


Figure 4.7 Variation of BOD and COD at the seven slaughterhouses, vertical bars are standard errors of the mean of n = 6.

The BOD and COD values were similar to the values reported in the literature such as 709.0 - 748.0mg/l for BOD and 340.0 - 1550.0mg/l for COD reported by Akan *et al.*, (2010); 135.00 ± 35.36mg/l for BOD by Ojo and Alamuoye (2015). Also Tidjanihisseine *et al.,* (2016) in their study of central abattoir in Moundou, Chad, reported that BOD ranged between 548mg/l in December and 614mg/l in May, while COD ranged between 109mg/l and 801mg/l. The high concentration of BOD and COD obtained from this study was expected because of the poor practice of blood and solids separation observed at most of the slaughterhouses. Blood was a main component of the sampled wastewater, and it is reported that blood is a major contributor of organic load with 150000mg/l to 200000mg/l BOD and 375000mg/l COD (Tritt, and Schuchardt, 1992). Therefore the high values from the slaughterhouses can mainly be attributed to the blood generated through the slaughtering operations. Lower values obtained at the Agulu slaughterhouse can be attributed to the blood pit provided at the slaughter slab for blood collection.

4.1.2.7 Nutrients

The average ammonium nitrogen and nitrate nitrogen concentrations were high at the slaughterhouses. For NH_4-N the mean values ranged from 52.0 mg/l at the Eke-Ekwulobia slaughterhouse to 107.5mg/l at the Ochanja slaughterhouse (Figure 4.8). NO_3-N values ranged from 34.5mg/l at the Amansea slaughterhouse to 58.5mg/l at the Ochanja slaughterhouse.

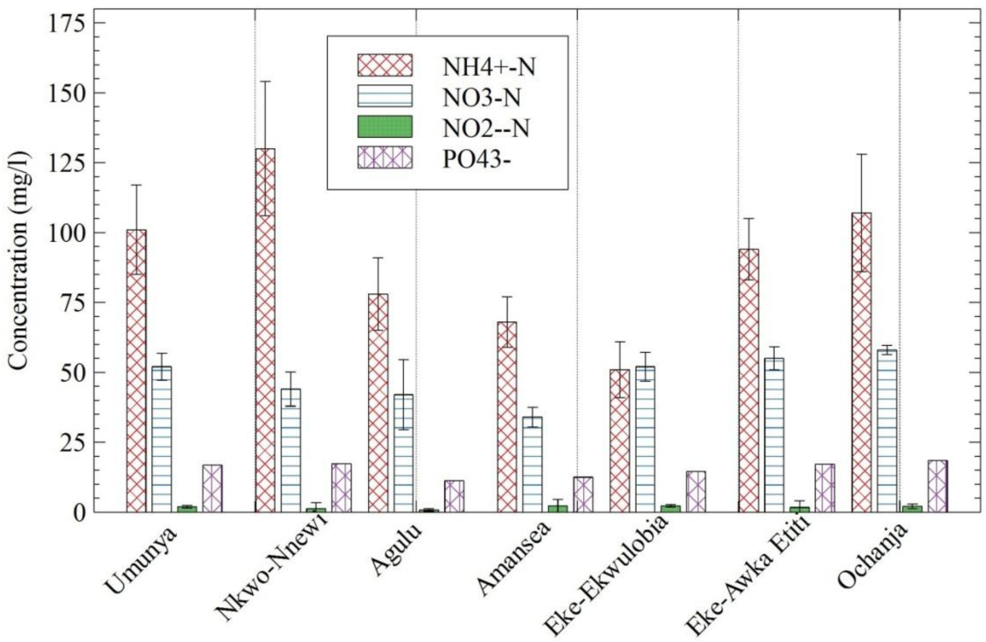


Figure 4.8 Variation of nutrients at the Seven slaughterhouses, vertical bars are standard errors of the mean of n = 6.

The NH_4-N and NO_3-N values recorded at the slaughterhouses were higher than FEPA (1991) limits of 10mg/l and 20mg/l respectively. These results were contrary to those reported by Ojo and Alamuoye (2015) with a NO_3-N range of 408.46 ± 3.50mg/l in effluents from the slaughter slab and 445.23 ± 2.25mg/l in drainages within the slaughterhouse. Also much higher NO_3-N values of 216.33 - 252mg/l were recorded by Ekanem *et al.,* (2016). However, Akan *et al.,* (2010) reported much similar results of 38 - 62mg/l for NO_3-N. Also the findings were similar to Tidjanihisseine *et al.,* (2016) who reported that concentration of NO_3-N for effluent from central abattoir, Chad ranged between 33.67mg/l in December and 53.1mg/l in May. The high NH_4-N and NO_3-N concentrations can be attributed to the richness of effluents in urine and organic matter. The levels of NH_4-N and NO_3-N in the slaughterhouse wastewater show that the wastewater can be treated by biological processes.

The mean NO_2-N concentrations were low at the slaughterhouses ranging from 0.84mg/l at the Agulu slaughterhouse and 2.27mg/l at the Amansea slaughterhouse. The low levels of NO_2-N found in wastewater studied can be attributed to the instability of nitrite ion (NO^-_2) due to oxidation reactions and also the fact that NO_2^- values are most often lower that the other forms of nitrogen related to it, which are NH_4^+ and NO_3^- (Noukeu *et al.,* 2016).

PO_4^{3-} mean values of the slaughterhouse effluents ranged from 11.2mg/l at the Agulu slaughterhouse to 18.5mg/l at the Ochanja slaughterhouse as shown in Figure 4.8. The concentrations recorded at the seven slaughterhouses were above FEPA (1991) limit of 5 mg/l for discharge into surface water. PO_4^{3-} levels were similar to the values reported by Akan *et al.,* (2010) with a range of 11 - 20 mg/l, but was not parallel with the works of Ojo and Alamuoye (2015) and Atuanya *et al.,* (2012) with higher values of 51.49 ± 3.31 mg/l and 24.5 - 32 mg/l respectively. Wu and Mittal (2011) in their Characterization of effluent from facilities Canada, also reported higher phosphate values of 86 mg/l for beef slaughterhouse, 35mg/l for pork, 34mg/l and 77mg/l for poultry and mixed use slaughterhouses respectively. The lower PO_4^{3-} levels may be as a result of low usage of detergents for washing the slaughter slab and equipment, as most of the butchers at the slaughterhouses washed their equipment with only water. The nutrients quantities in the slaughterhouse effluent can lead to eutrophication when discharged into water bodies, which can kill aquatic life because of oxygen depletion as result of excessive algae growth and mineralization of dead algae.

As can be seen in the graphs, the mean values of most physicochemical wastewater parameters were lowest at the Agulu slaughterhouse compared to the wastewater from the other slaughterhouses. The difference between the mean physicochemical parameter values of the Agulu slaughterhouse wastewater and the mean values from the other slaughterhouses were viewed statistically to see if they varied significantly of not. One-way Multivariate analysis showed that there was a statistically significant difference in the mean physicochemical parameters of the water samples collected at the different slaughterhouses, $F(72, 136.4) = 4.42$, $p = 0.000$, *Wilk's* $\wedge = 0.01$, $\eta^2 = 0.664$. Multiple comparison using Tukey post hoc test revealed that the mean pH of the Agulu slaughterhouse wastewater was significantly lower ($p < 0.05$) than that of the Nkwo-Nnewi, Eke-Ekwulobia, Eke-Awka Etiti and Ochanja slaughterhouses. There difference between the mean pH of Agulu and Umunya slaughterhouse wastewater was not statistically significant ($p = 0.992$), as well as Agulu and Amansea slaughterhouse wastewater ($p = 0.998$). There was no statistically significant difference ($p > 0.05$) between the mean EC of the wastewater from the Agulu slaughterhouse and that of the other slaughterhouses. There was also no statistically significant difference ($p > 0.05$) between the mean temperature and TDS of the wastewater from the Agulu slaughterhouse and that of the other slaughterhouses. The difference between the wastewater mean TSS of Agulu and Eke-Ekwulobia slaughterhouses was not statistically significant ($p = 0.971$), however, the wastewater mean TSS at the former significantly differed from that of the Umunya ($p = 0.01$), Nkwo-Nnewi (*p = 0.037*), Amansea (*p = 0.052*), Eke-Awka Etiti (*p = 0.013*) and Ochanja (*p = 0.000*) slaughterhouses. The mean value of TS of the wastewater from the Agulu slaughterhouse significantly differed (*p = 0.014*) from that of the Ochanja slaughterhouse, but did not differ significantly (*p > 0.05*) from the mean values of wastewater from the other slaughterhouses. Apart from the Ochanja slaughterhouse wastewater that had a significantly higher BOD (*p = 0.034*) and COD (*p = 0.009*) mean values compared to the mean value of the Agulu slaughterhouse wastewater, the BOD and COD of the wastewater did not significantly differe at the slaughterhouses. The NH_4-N mean value of the Agulu slaughterhouse wastewater was significantly lowere compared to that of Nkwo-Nnewi (*p = 0.000*) and Ochanja (*p = 0.043*) slaughterhouses. The NO_3-N mean value of the Agulu slaughterhouse wastewater was significantly lowere compared to that of Eke-Awka Etiti (*p = 0.013*) and Ochanja (*p = 0.001*) slaughterhouses. The NO_2-N mean value of the Agulu slaughterhouse wastewater was significantly lowere compared to that of Amansea (*p =0.011*), Eke-Ekwulobia (*p = 0.021*) and Ochanja (*p = 0.031*) slaughterhouses. The PO_4^{3-}mean value of the Agulu slaughterhouse wastewater was significantly lowere compared to that of Umunya (*p = 0.001*), Nkwo-Nnewi (*p = 0.000*), Eke-Awka Etiti (*p = 0.000*) and Ochanja (*p = 0.000*) slaughterhouses.

The significantly lower concentrations of most of the physicochemical wastewater parameters recorded in the Agulu slaughterhouse can be attributed to the fact that a blood pit was provided at the slaughter slab and thus blood was collected separately, unlike in the other slaughterhouses were it was allowed to flow down effluent channel. Blood is known to significantly increase the organic load of wastewater and the efficiency of its collection during animal slaughtering is very critical in reducing the oxygen demand of the effluent (Tritt and Schuchardt, 1992). The high variation in physicochemical parameters can also be attributed to the variation in the number of animals slaughtered at the facilities, as the number of cows slaughtered at the Ochanja slaughterhouse was 8 times the number slaughtered at the Agulu slaughterhouse. While the

Agulu slaughterhouse dealt exclusively on cattle, some of the other slaughterhouses were mixed use slaughterhouses for cattle, goat and sheep, thus more wastewater with higher strength was expected.

A Pearson product-moment correlation was carried out to determine the relationship amongst physicochemical parameters of wastewater from Agulu slaughterhouse. This was done to give a greater understanding of the quality of slaughterhouse wastewater and the possibility of predicting parameter concentrations. The correlation analysis showed expected trends in wastewater quality. The result revealed that there was a strong, positve and statistically significant correlation between EC and TDS ($r = 0.829, p = 0.041$); EC and TS ($r = 0.942, p = 0.005$); EC and COD ($r = 0.956, p = 0.003$); EC and BOD ($r = 0.979, p = 0.001$); TDS and TS ($r = 0.950, p = 0.004$); TS and COD ($r = 0.880, p = 0.021$); TS and BOD ($r = 0.863, p = 0.027$); COD and BOD ($r = 0.972, p = 0.001$). The correlations of the other physicochemical parameters were generally moderate to weak and were not statistically significant ($p > 0.05$). The very strong correlation observed between COD and BOD is in line with the report of Akan *et al.*, (2010) who observed a positive linear relationship between BOD and COD concentrations. Due to the importance of organics in wastewater treatment and the high costs associated with there laboratory determination, COD was regressed against BOD. The result showed that the regression model predicts COD significantly well, thus, prediction of COD values from BOD results and vice verse, can be done using the regression equation and can be relatively reliable. However, Abdalla and Hammam (2014) opined that "correlation should be periodically rechecked due to probable seasonal variations in climatic conditions, social customs, water supply characteristics, water availability, population size, or the presence of other wastes".

The average Biodegradability Index (B.I) for wastewater from the seven slaughterhouses is presented in Table 4.3.

Table 4.3 Biodegradability index for the different slaughterhouses

Slaughterhouse	Biodegradability Index
Umunya	0.70
Nkwo-Nnewi	0.57
Amansea	0.66
Eke-Ekwulobia	0.58
Agulu	0.60
Eke-Awka Etiti	0.63
Ochanja	0.62

The result revealed that wastewater from most of the slaughterhouses can be effectively treated by biological means. This in line with Sunder and Satyanarayan, (2013) who stated that slaughterhouse depicts BOD/COD ratio of 0.6 indicating its highly biodegradable nature. The average B.I of 0.61 obtained in this study was higher than the typical value of 0.5 for raw domestic wastewater.

4.2 Pilot Horizontal Subsurface Flow Constructed Wetland Results

Most research on the growth and treatment response of constructed wetland macrophytes had tertiary treatment of domestic wastewater as their main concern. This study focused on the secondary treatment of slaughterhouse effluent using HSSF CW, with emphasis on the performance of three locally available plants and the use PKS as a substrate material. In this section, the growth of the plants and their performance in the pilot HSSF CW treating slaughterhouse wastewater are reported. The suitability of PKS for use in the system is also reported.

4.2.1 Growth Characteristics of Macrophytes in Slaughterhouse Wastewater

At start-up, the acclimatization of *Typha latifolia* was very fast, with emerging juvenile shoots appearing lush-green. The *Colocasia esculenta* appeared stressed at start-up, but gradually became more lush green and healthier after the establishment period. During the growing period, *Typha latifolia* and *Thalia geniculata* showed development cycles characterized by periods of rapid exponential growth, followed by a lag phase lasting around 10 weeks, as opposed to *Colocasia esculenta* shoots whose progress started in June and gradually increased before hitting a lag phase in September. Shoot height increased rapidly for *Typha latifolia* and *Thalia geniculata* compared to *Colocasia esculenta* with the macrophytes reaching heights of 2.1m, 1.4m, and 0.7m respectively within 3 months of establishment as shown in Figure 4.9. *Colocasia esculenta* increased in height with increasing time.

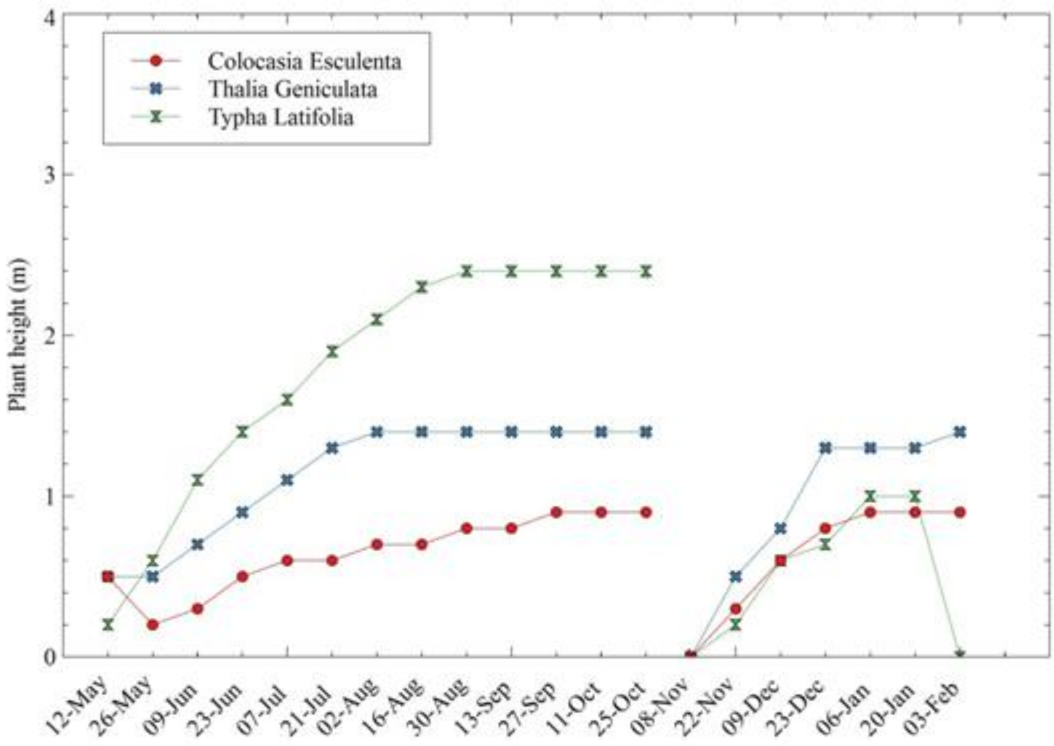


Figure 4.9 Variation of shoot heights during the experimental period (Shoots were harvested in November 2016 giving way to new growth).

The *Typha latifolia* first flowering occurred about four months after the transplant. Heights of 1.0m, 1.4m and 0.9m were attained for *Typha latifolia*, *Thalia geniculata* and *Colocasia esculenta* respectively after harvest.

From a density of 28.6 shoots/m^2 at the start of the study, 80 shoots/m^2 were obtained after 3 months of establishment for the cell planted with *Typha latifolia*, 97.1shoots/m^2 for the cell planted with *Thalia geniculata* and 57.1 shoots/m^2 for the cell planted with *Colocasia esculenta*. Following batch feeding with slaughterhouse wastewater, abundance increased to 137.1 shoots/m^2 for the cell planted with *Typha latifolia*, 211.4 shoots/m^2 for the cell planted with *Thalia geniculata* and 97.1 shoots/m^2 for the cell planted with *Colocasia esculenta* before harvest as shown in Figure 4.10.

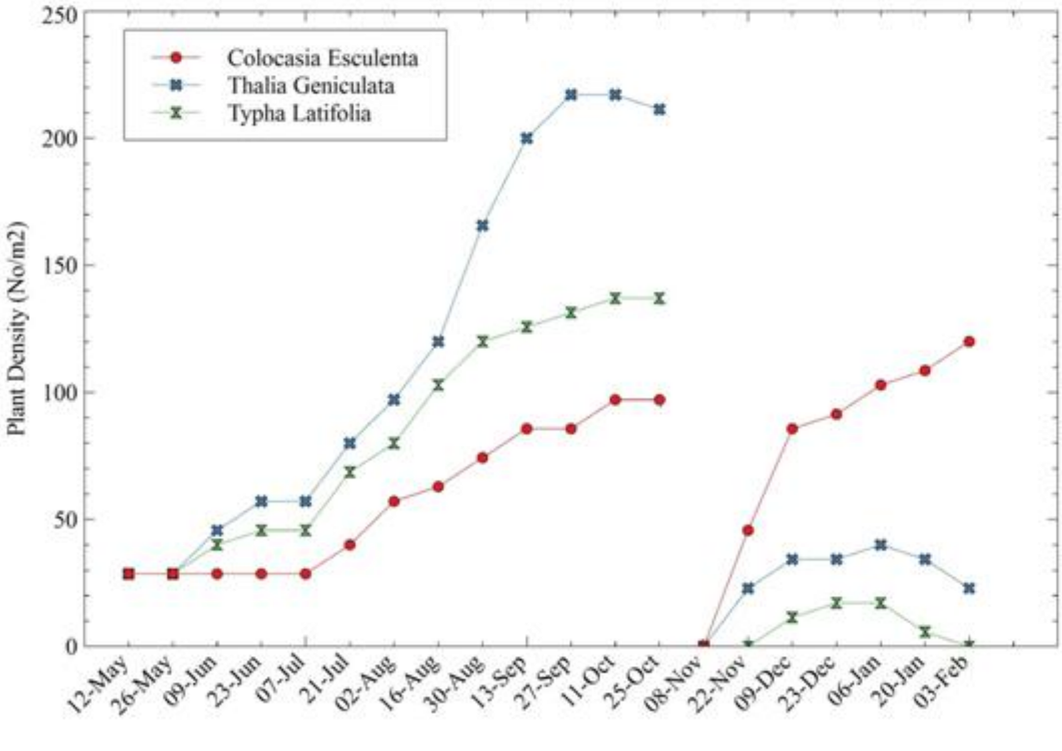


Figure 4.10 Variation of shoot density during the study period (Shoots were harvested in November 2016 giving way to new growth).

The increase was slower in the subsequent 3 months after harvest for *Thalia geniculata*, with a maximum abundance of 40shoots/m^2. *Typha latifolia* was stressed after the harvest, attaining a maximum density of 17.1shoots/m^2 before all the shoots weathered and died. *Colocasia esculenta* continued to increase in numbers reaching 120shoots/m^2 in the same period. The above-ground biomass yield for *Thalia Geniculata* was the highest with a value of 4.57kg dry biomass/m^2. The values of the above-ground biomass obtained for *Typha latifolia* and *Colocasia esculenta* were 2.86 and 1.71kg dry biomass/m^2 respectively. The macrophyte species used in this study have been shown to survive and reproduce well in slaughterhouse wastewater.

The maximum shoot height recorded for *Typha latifolia* was within the ranges (2 - 2.5m) for mature plants in the source natural wetland. The maximum shoot height recorded for *Thalia geniculata* was lower than the range (1.6 - 1.8m) for mature plants in the source natural wetland. Also, for *Colocasia esculenta* the maximum shoot height was much lower than the range (1.2 - 1.5m) for the source natural wetland. The lower shoot heights obtained for *Thalia geniculata* and *Colocasia esculenta* compared to the values obtained from the source natural wetlands can be attributed to the influence of the high strength slaughterhouse wastewater on the plants, which is similar to the submissions of Nagajyothi *et al.*, (2009) that decline in growth of macrophytes with increasing concentration of effluent.

Statistical analysis (one-way ANOVA) showed that there was a significantly higher mean plant densities of *Thalia geniculata* than that of *Typha latifolia* ($p = 0.000$). The mean difference between the shoot densities of *Thalia geniculata* and *Colocasia esculenta* was not significant ($p = 0.19$). The difference in the plant densities for *Typha latifolia* and *Colocasia esculenta* was also not significant ($p = 0.54$). However, after harvest, the shoot density of *Colocasia esculenta* was found to be significantly higher ($p = 0.002$) than that of *Thalia geniculata*, and also significantly higher ($p = 0.000$) than that of *Typha latifolia*. Thus shoot regeneration rate was highest with *Colocasia esculenta.*

If the plants overall growth performance were to be based on shoot heights and densities, it would be misleading because there are other factors, such as size of leaves, that should be incorporated in defining the overall plant growth. Thus biomass gain is regarded as a better parameter to measure overall plant productivity (Luo and Rimmer, 1995). The above-ground biomass yield for *Thalia geniculata* was approximately 1.6 times that of *Typha latifolia* and approximately 2.7 times that of *Colocasia esculenta.* Thus *Thalia geniculata* had the highest productivity amongst the three macrophytes studied. Higher biomass yield obtained for *Thalia geniculata* was similar to the findings of Polomski *et al*., (2008), in their study of differential nitrogen and phosphorous recovery by five aquatic garden species in a laboratory-scale subsurface constructed wetland, who reported a higher rate of dry weight accumulation for *Thalia geniculata*, compared to *Typha spp.*, which they attributed to the fact that *Thalia geniculata* were supplied with greater amounts of N and P than the other species as a result of their higher evapotranspiration rate.

The above-ground biomass obtained for *Typha latifolia* in this study was close to the value obtained (1.5kg dry biomass/m^2) in a system for domestic effluent remediation (Kouki *et al*., 2012). The biomass yield of *Typha latifolia* was consistent with the average value of 3kg dry biomass/m^2 reported by Chong *et al.*, (2009). However, Vymazal (2011) in their review of plants used in subsurface flow constructed wetland reported that *Typha latifolia* is a very productive species with maximum above-ground biomass values in both natural stands and constructed wetlands exceeding 5 kg dry biomass/m^2, which indicated the values obtained in this study were below the expected optimum productivity of the plant. The biomass yield of *Colocasia esculenta* was the lowest but the low biomass yield can be compensated by the observed increase in the shoot density after the harvest, which may translate to a higher biomass yield at some point. Also worthy of note is the fact that *Colocasia esculenta* appeared to be the most resilient to the extreme weather conditions in the dry season with less dead plants.

4.2.2 Effect of PKS on Macrophyte Growth and Development

Few days after planting the macrophytes in the pilot wetlands, the *Typha latifolia* planted in PKS were observed to be wilted and by the third week they were dead, while the *Typha latifolia* in gravel had grown to a height of 0.5m as shown in Figure 4.11. This is likely due to the presence of residual palm oil in the pilot PKS wetland cell, as was also observed by Chong *et al.*, (2009) for some other macrophytes planted in PKS. So the study on the feasibility of PKS as substrate was limited to pilot wetland cell planted with *Thalia geniculata* which showed very good resistance to the effects of residual palm oil in the PKS cell.

(a) (b)

Figure 4.11 *Typha Latifolia* after three of planting: (a) in PKS; (b) in Gravel.

Prior to the experimental treatments, the *Thalia geniculata* had grown into thick vegetation. Increase in shoot height was faster in the gravel cell compared to the PKS cell with macrophytes reaching maximum heights of 1.4m and 1.1m respectively within 3 months of establishment. The variation in plant heights during the study period is shown in Figure 4.12.

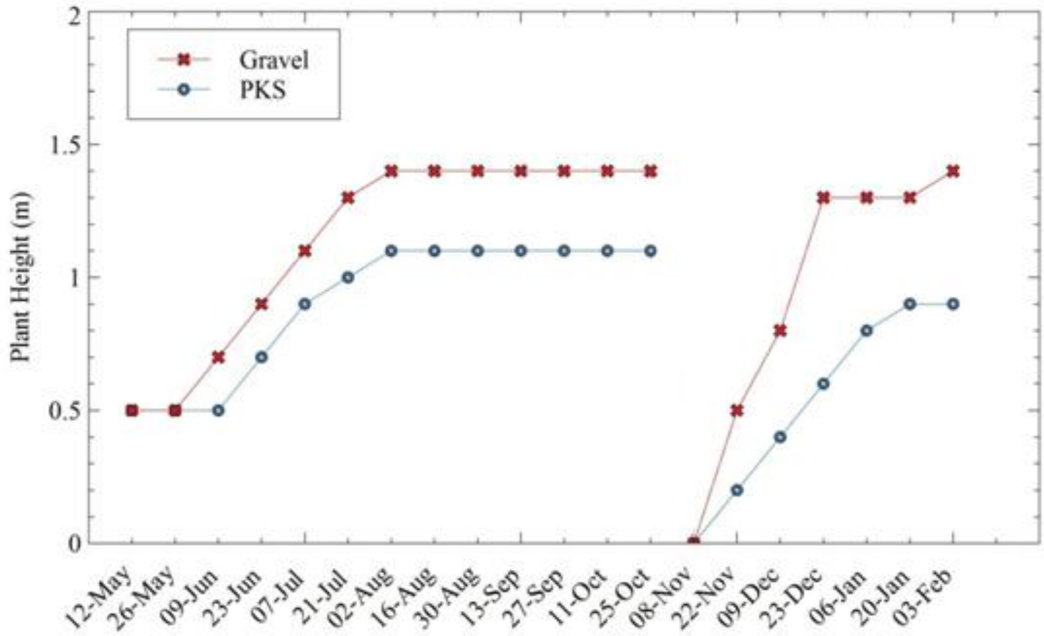


Figure 4.12 Variation in shoot heights in the gravel and PKS media during the experimental period (Shoots were harvested in November 2016 giving way to new growth).

The difference in height of the macrophytes in the gravel and those in PKS after three months of establishment can be clearly seen in Figure 4.13.

Figure 4.13 Difference in height of the *Thalia geniculata* in gravel and PKS substrate

From a density of 28.6shoots/m^2 at start of the study, 97.1shoots/m^2 was obtained after 3 months of establishment for the gravel substrate and 62.9shoots/m^2 for the PKS substrate (Figure 4.14).

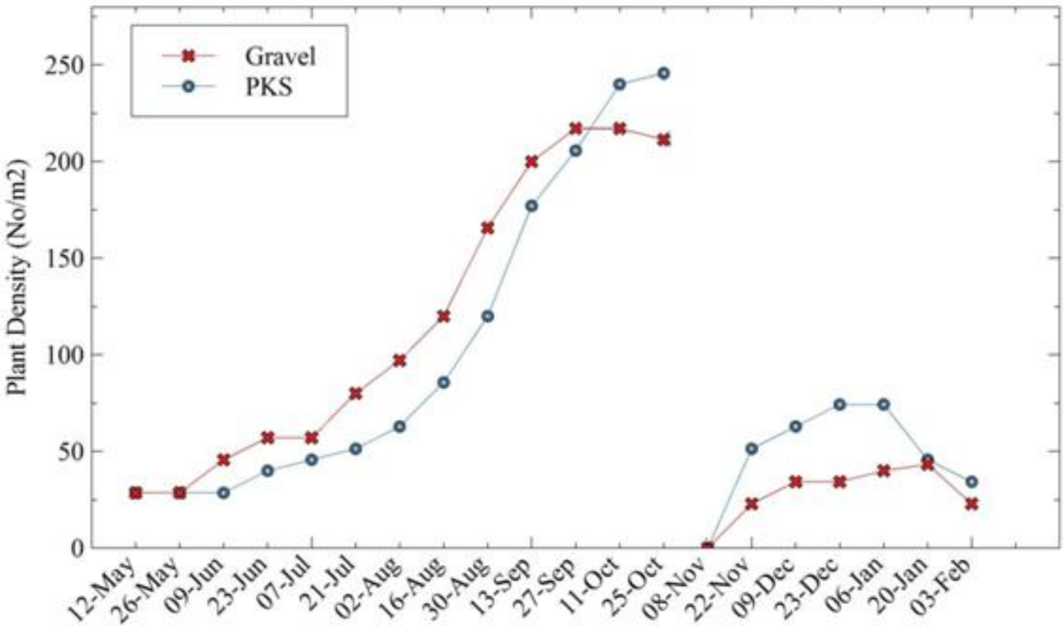


Figure 4.14 Variation in shoot density in the gravel and PKS during the study period (Shoots were harvested in November 2016 giving way to new growth).

Abundance increased with time following batch feeding with slaughterhouse wastewater. Prior to harvest, the abundance in the PKS had surpassed that of gravel with values of 245.7shoots/m^2 and 211.4shoots/m^2 respectively. The value of the above-ground biomass yield for *Thalia geniculata* in the PKS cell was 4.72kg dry biomass/m^2. The value was a little higher than the biomass yield of 4.57kg dry biomass/m^2 obtained for the *Thalia geniculata* in gravel.

The results revealed that macrophyte height was influenced by the type of substrate used, which is in line with the submissions of Chong *et al.*, (2009) who stated that the height of macrophytes was affected by the micro-ecosystem of their substrate. However, their observation of higher plant growth in PKS than in gravel was contrary to the findings of this study with increased height in the gravel than the PKS substrate. The mean difference between the shoot densities of *Thalia geniculata* in gravel and PKS was not significant ($p = 0.92$). Slower rate of increase in abundance of plants in the PKS substrate can be attributed to the effects of residual palm oil in the cell, which cleared with the batch loading of wastewater resulting in higher abundance towards the harvest period. Chong *et al.*, (2009) also reported that PKS based microcosm gave a higher shoot generation rate than gravel. Higher dry biomass yield obtained for PKS substrate was consistent with the submissions of Chong *et al.*, (2009) that the rate of dry biomass gain was dependent on the medium in the microcosm and the stage of plant growth, and that biomass gain in the PKS based microcosm was higher with an average rate of 4.4 kg dry biomass/m² as against the 3 kg dry biomass/m² for the gravel based media.

4.2.3 Hydrology of the Pilot Wetlands

The water balance of the pilot cells was dominated by wastewater inflows and outflows as rainwater was not allowed to enter the cells. The inflow was not constant due to the increase in the below ground biomass with time. The outflows were also variable due to evapotranspiration (ET) losses. The summary data is shown in Table 4.4. The mean ET rates that were determined over a total period of seven months were as follows: control - 1.21 mm/day; *Typha Latifolia* - 1.51 mm/day; *Thalia Geniculata* - 1.58 mm/day and *Colocasia Esculenta* - 1.40 mm/day. The outflows from all the wetland cells were found to be significantly lower than the inflows ($p < 0.05$).

Table 4.4 Summary data of mean inputs and outputs to each of the wetland cells (n = 14)

			Wetland Cells		
	Unit	Control	*Typha Latifolia*	*Thalia Geniculata*	*Colocasia Esculenta*
Input Wastewater Inflow	m^3	0.012	0.0088	0.0079	0.0085
	S.E	0.00051	0.0013	0.0014	0.0084
Output Effluent Outflow	m^3	0.0086	0.0051	0.0041	0.0050
	S.E	0.0008	0.002	0.002	0.005
ET	mm/day	1.21	1.51	1.58	1.43
	S.E	0.12	0.18	0.21	1.46

S.E = Standard Error

ET contributes considerably to the hydraulic load (Kadlec and Knight, 1996). It accounted for 28.3%, 42.0%, 48.1% and 41.2% of outputs from the control, *Typha latifolia*, *Thalia geniculata* and *Colocasia esculenta* cells respectively. The ET rates of *Thalia geniculata* was significantly different from that of *Typha latifolia* ($p = 0.\ 0007$) and *Colocasia esculenta* ($p = 0.000$). Expectedly, ET rate was significantly lower ($p < 0.05$) in the control than the planted cells.

ET rate was highest in the cell planted with *Thalia geniculata*, which suggested that macropytes type was a major contributing factor affecting the ET rates. The higher ET rate can be attributed to the fact that the *Thalia geniculata* had highest number of shoots and biomass, which invariably led to more ET losses. Polomski *et al*., (2008) also reported higher ET rates with *Thalia geniculata* compared to other macrophytes. However, it was contrary to the conclusions of Bernatowicz *et al.*, (1976) and Koerselman and Beltman (1988), which reported that the type of vegetation does not significantly influence ET rates. Amongst all the planted wetlands, the mean daily ET rates were lowest in the *Colocasia esculenta* cells, which can be attributed to the low leaf area index due to the plants poor start-up. The control cell had the lowest ET rates, which underscored the importance of wetland macrophytes in constructed wetland systems.

Abtew and Obeysekera (1995) in their lysimeter study of *Typha domingensis* in South Florida reported an average measured ET rate of 3.9mm/day. Abtew (1996) in their study to measure and model ET in three wetland systems also reported an average ET rate of 3.6mm/day for cattails. Kato *et al*., (1969) in their study of the characteristic features of water consumption of various crops in Japan reported that the daily mean ET rates for *Colocasia esculenta* were in the range of 5 - 7mm/day in the summer. Mangistu *et al*., (2014) studied the ET rate of *Colocasia esculenta* and Sedge in South Africa and reported that ET rates ranged from 1 - 6mm/day, with daily average ET rate of 3.5mm/day in November 2009 and 3.3mm/day in January 2010. Data on ET rates from CWs in the humid tropics is lacking and more especially in West Africa.

However, exceedingly high ET rates can also be expected due to the prevailing intense solar radiation (Kadlec and Knight, 1996). The relatively lower ET rates (1.21 - 1.58mm/day) estimated from this study compared to some of the literature values can be attributed to a number of factors such as the small size of the cells and differences in relative humidity at the differnt sites. Higher values reported in the literature can also be attributed to the fact that the depth of water is constant in the most CW beds due to continuous flow operations that is obtainable in most systems, unlike the batch flow regime implemented in this study where water levels declined continuously without replacement to compensate for ET losses, as it is a well known fact that ET depends on water availability.

The mean daily ET rates were 1.58mm/day and 1.37mm/day for the *Thalia geniculata* planted in gravel and PKS respectively. The ET rate from the gravel bed was significantly higher than the PKS bed ($p < 0.05$). The lower ET rates observed in the PKS bed can be attributed to slower rate of increase in abundance of plants in the PKS substrate due to the effects of residual palm oil in the bed. However, the ET rates increased with time, almost equaling that of the gravel bed. This can also be attributed to the fact that the residual oil in the bed cleared with the batch loading of wastewater resulting in higher abundance, and invariably higher ET rates. The significantly higher ET rates from the gravel cell suggested that the media type in the wetlands influence the water loss to the atmosphere. However, there were variations in the plant characteristics such as height, leaf area index and biomass yield in the two wetland cells, thus drawing conclusions from the above may be misleading. Data on the ET rates from PKS based CWs in the humid tropics is also lacking.

4.2.4 Pollutants Removal Performance of Macrophytes in the Pilot Wetlands

The concentration of the influent pollutant parameters varied throughout the study period with a range of 511.3 - 906.8mg/l for BOD; 182.4 - 305.1mg/l for TSS; 43.6 - 81.0mg/l for NH_4-N; 27

- 51mg/l for NO_3-N and 5 - 13mg/l for PO_4^{3-}. These variations were similar to the observed variability of pollutant concentration in slaughterhouse wastewater in the study site, due to variations in number of animals slaughtered daily, the daily water usage and sampling time. The pollutant removal efficiencies of the cells were measured in terms of mass reduction percentages instead of percentage concentration reduction. This was necessary because important differences were observed for water loss in all cells due to ET, which led to higher pollutant concentrations in the effluent collected from the planted and unplanted wetland cells. Therefore, evaluation of mass removal rates and percentages were more appropriate means of comparing the performance of the different plant species.

4.2.4.3 BOD Removal

Effluent BOD concentrations showed very strong fluctuation. Effluent BOD concentration ranged from 333.4 - 656.5mg/1 for control cell; 121.2 - 768.4mg/1 for the *Thalia geniculata* cell; 274.8 - 560.4mg/1 for the *Colocasia esculenta* cell; and 162.3 - 685.8mg/1 for the *Typha latifolia* cell. Table 4.5 shows the BOD influent and effluent areal loading rates.

The influent mass loading rate for the control cell ranged between 32.12 - 62.64g/m^2.batch with a mean value of 49.27 ± 8.1g/m^2.batch and mass removal rates (MRRs) varying between 13.50 - 43.88g/m^2.batch with a mean value of 29.71 ± 8.8g/m^2.batch. The influent mass loading rate for the *Thalia geniculata* cell ranged between 19.27 - 46.28g/m^2.batch with a mean value of 33.75 ± 7.4g/m^2.batch and mass removal rates varying between 11.46 - 41.64g/m^2.batch with a mean value of 25.15 ± 7.7g/m^2.batch. The influent mass loading rate for the *Typha latifolia* cell ranged between 21.61 - 49.0g/m^2.batch with a mean value of 37.23 ± 7.5g/m^2.batch and mass removal rates varying between 15.64 - 42.45g/m^2.batch with a mean value of 29.78 ± 7.8g/m^2.batch. The influent mass loading rate for the *Colocasia esculenta* cell ranged between 22.78 -

47.63g/m^2.batch with a mean value of 36.23 ± 6.4g/m^2.batch and mass removal rates varying between 14.71 - 37.69g/m^2.batch with a mean value of 25.45 ± 5.7g/m^2.batch.

Table 4.5 BOD mass statistics for influent and effluent of the pilot wetland cells

		BOD					
	Mass Loading (g/m^2.batch)	Min	Max	Mean	SD	MRR	RE (%)
Control	Influent	32.12	62.64	49.27	8.13		
	Effluent	16.07	28.11	19.55	3.01	29.71	59.14
Thalia geniculata	Influent	19.27	46.28	33.75	7.48		
	Effluent	4.63	11.42	8.07	2.26	25.15	75.42
Typha latifolia	Influent	21.61	49.00	37.23	7.59		
	Effluent	4.78	11.77	7.44	1.93	29.78	79.19
Colocasia esculenta	Influent	22.78	47.63	36.23	6.42		
	Effluent	8.06	14.45	10.78	1.95	25.45	69.81

MRR is the Mass removal rate and RE is the Removal efficiency

Figure 4.15 and 4.16 presents the influent and effluent BOD areal loading and mass removal efficiencies for the different cells during the experimental run. The average mass removal efficiency for the control, *Typha latifolia*, *Thalia geniculata* and *Colocasia esculenta* cells were 59.1%, 79%, 75.4%, and 69.8% respectively.

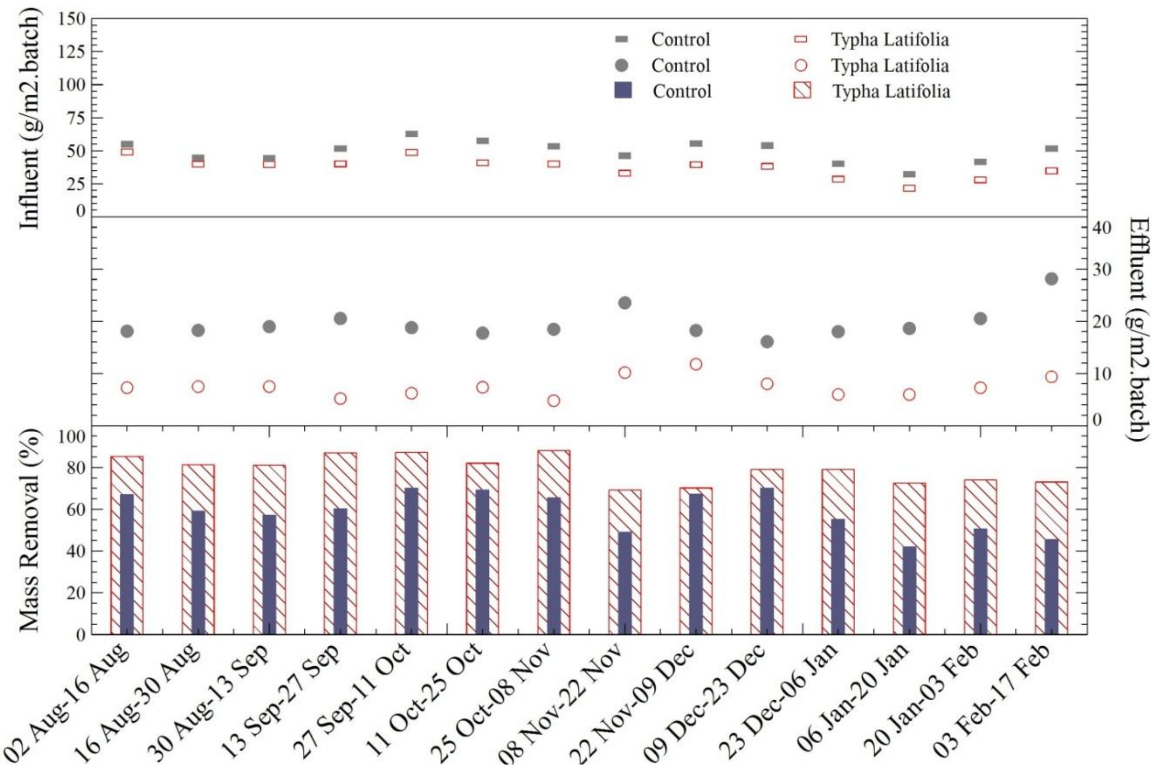


Figure 4.15 Influent and effluent BOD areal mass loading and mass removal efficiencies of the control and *Typha latifolia* cells

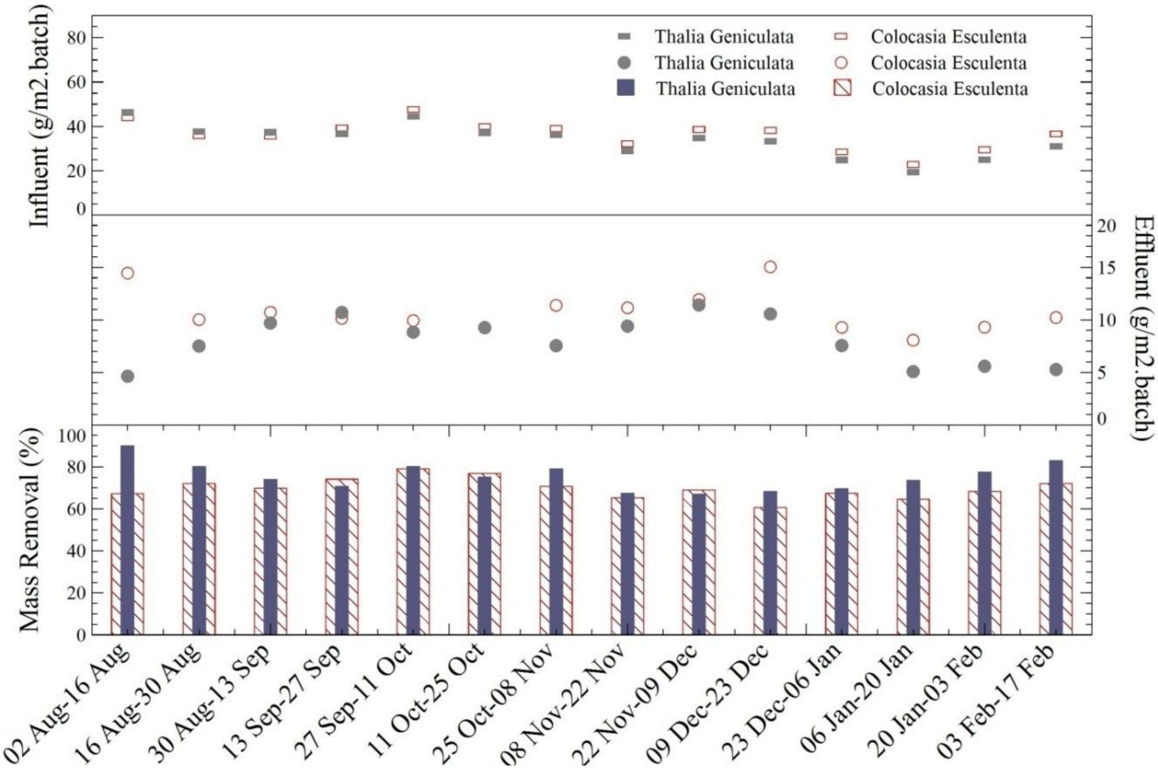


Figure 4.16 Influent and effluent BOD areal mass loading and mass removal efficiencies of the *Thalia geniculata* and *Colocasia esculenta* cells

High BOD removal rates were observed in the macrophyte cells and the values recorded were similar to the ranges reported in the literature. Reported treatment efficiencies by constructed wetlands were 82.5% (Adeniran *et al.,* 2014); 71 ± 6.2% (Mairi *et al.,* 2012); 82-85% (Badejo *et al.,* 2012), 74% (Kadlec and Knight, 1996) and 60-61% (Ansola *et al*., 2003).

There was a statistically significant difference between the mass removal efficiency of the four cells as determined by one-way ANOVA ($p = 0.000$). A Tukey post hoc test revealed that the unplanted cell was statistically significantly outperformed by the planted cells ($p = 0.00$ for *Thalia geniculata;* $p = 0.00$ for *Typha latifolia*; $p = 0.01$ for *Colocasia esculenta*). The mean difference in the removal efficiency between the cells with macrophyte and the control was 15.7%. The significant difference found between the BOD removal efficiency of the wetlands suggested that the plants provided an important practical benefit towards the wetland system performance with respect to organic matter elimination. Korboulewsky *et al.*, (2012) stated that the crucial purification role of plants is as a result of the large surface area plant roots provide for microbial activities. Increased BOD removal from wastewater has been reported in macrophytes root region and rhizosphere where higher dissolved oxygen concentrations stimulate pollutant degradations (Brix, 1997; Sim, 2003). This is in tandem with the statement of Gersberg *et al.,* (1986) that better removal of organic carbon occur in the rhizosphere due to translocation of oxygen. Significant differences in the organic matter removal efficiency of beds with macrophytes and control have also be recorded in several studies (Brix, 1997; Stottmeister *et al*., 2003; Vymazal, 2011; Brisson and Chazarenc, 2009)

Tukey post hoc test also revealed that the mass removal efficiency was statistically higher ($p = 0.05$) in the cell with *Typha latifolia* compared to the cell with *Colocasia esculenta.* However, the difference between the cells with *Typha latifolia* and *Thalia geniculata* was not statistically significant ($p = 0.502$). As well as between the *Thalia geniculata* and *Colocasia esculenta* cells (*p = 0.169*). The higher removal efficiency of the *Typha latifolia* cells compared to the *Colocasia esculenta* can be as a result of the differences in plant root penetration. *Typha latifolia* roots developed a dense mat compared to that of the *Colocasia esculenta*. However, the higher removal rate should be taken rather conservatively due to the insignificant difference between the *Thalia geniculata* and *Colocasia esculenta* cells. This conclusion was based on the fact that the shoot density and above ground biomass of *Thalia geniculata*, which could have been contributing factors to the mass removal efficiencies of the cell, and which was higher compared to *Typha latifolia*, did not result to a significantly higher removal efficiency compared to *Colocasia esculenta*. Correlation analysis showed a strong positive relationship between the influent loading rates and the corresponding MRRs for all the four cells. The MRRs increased proportionally with the ILRs. Regression analysis revealed a high predictability of the beds treatment efficiency as regards the mass removal of organic matter, with regression coefficient (R^2) ranging from 0.86 for the control cell to 0.94 for the *Typha latifolia* cell.

The observed relationship between influent loading rate and mass removal rate followed first-order kinetics, thus removal ratee were proportional to the inflow amount. This is in line with the reported relationship in the literature. Tanner *et al*., (1995) in their study of the influence of mass loading on treatment performance of wetlands reported that there was no significant difference in the performance of the grave bed with macrophyte and the control bed for BOD removal, but that there was higher removal efficiency for BOD when the mass loading increased. Organic loads higher than 67kg BOD $ha^{-1}d^{-1}$ (Metcalf and Eddy, 1991) or out of the range of 67 - 157kg BOD $ha^{-1}d^{-1}$ (USEPA, 1999) is not recommended for HSSF CW. For both batch and continuous flow systems, WPCF, (1990) recommended a maximum loading of 100kg BOD $ha^{-1}d^{-1}$. The results from this study indicated that the HSSF CW cells had the capacity to handle higher loads than applied in the study, which will be facilitated by high temperatures in the study area.

4.2.4.2 TSS Removal

Effluent TSS concentration ranged from 46.6 - 127.0mg/1 for control cell; 12.8 - 179.1mg/1 for the *Thalia geniculata* cell; 35.1 - 142.1mg/1 for the *Colocasia Esculenta* cell and 19.7 - 128.1mg/1 for the *Typha latifolia* cell. Table 4.6 presents the TSS influent and effluent areal loading rates.

Table 4.6 TSS mass statistics for influent and effluent of the pilot wetland cells

	Mass Loading (g/m^2.batch)	TSS					
		Min	Max	Mean	SD	MRR	RE (%)
Control	Influent	11.96	21.09	15.79	2.95		
	Effluent	2.21	5.86	4.07	1.23	11.72	74.19
Thalia geniculata	Influent	7.49	16.38	10.85	2.83		
	Effluent	0.46	1.59	1.15	0.40	9.52	88.18
Typha latifolia	Influent	8.50	17.34	11.96	2.86		
	Effluent	0.64	3.26	1.97	0.84	9.99	82.74
Colocasia esculenta	Influent	8.32	16.03	11.65	2.53		
	Effluent	1.26	4.44	2.89	0.94	8.76	75.23

MRR is the Mass removal rate and RE is the Removal efficiency

The influent TSS mass loading rate for the control cell ranged between 11.96 - 21.09g/m^2.batch with a mean value of 15.79 ± 2.9g/m^2.batch and mass removal rates varying between 8.62 - 15.35g/m^2.batch with a mean value of 11.72 ± 2.5g/m^2.batch. The influent TSS mass loading rate for the *Thalia geniculata* cell ranged between 7.49 - 16.38g/m^2.batch with a mean value of 10.85 ± 2.83g/m^2.batch and mass removal rates varying between 5.53 - 15.91g/m^2.batch with a mean value of 9.52 ± 3.1g/m^2.batch. The influent TSS mass loading rate for the *Typha latifolia* cell ranged between 8.50 - 17.34g/m^2.batch with a mean value of 11.96 ± 2.8g/m^2.batch and mass removal rates varying between 6.34 - 14.27g/m^2.batch with a mean value of 9.99 ± 2.88g/m^2.batch. The influent TSS mass loading rate for the *Colocasia esculenta* cell ranged between 8.32 - 16.03g/m^2.batch with a mean value of 11.65 ± 2.53g/m^2.batch and mass removal rates varying between 6.27 - 11.88g/m^2.batch with a mean value of 8.75 ± 1.9g/m^2.batch.

Figure 4.17 and 4.18 presents the influent and effluent TSS areal loading and mass removal efficiencies for the different cells during the experimental run. The average TSS mass removal efficiency were 74.2%, 88.1%, 82.7% and 75.2% for control, *Thalia geniculata*, *Typha latifolia* and *Colocasia esculenta* cells respectively. The TSS removal rates achieved by the cells were consistent with ranges reported in the literature such as 86% by Molle *et al*. (2004) for 54 CWs in France and 80.01% by Nzabuheraheza *et al.,* (2012). However, Dhulap and Patil (2014) in their evaluation of the use of *Pennisetum purpureium* for the remediation of sewage reported a lower removal efficiency of 55.17%.

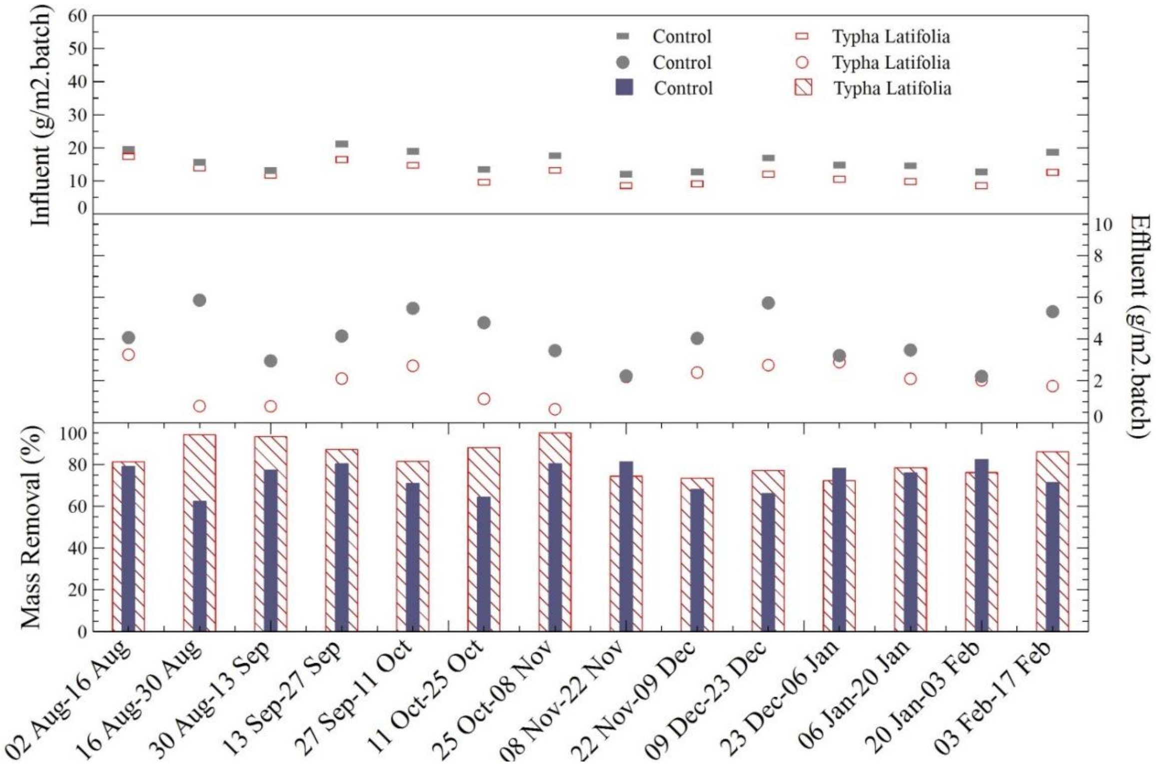


Figure 4.17 Influent and effluent TSS areal mass loading and mass removal efficiencies of the control and *Typha latifolia* cells

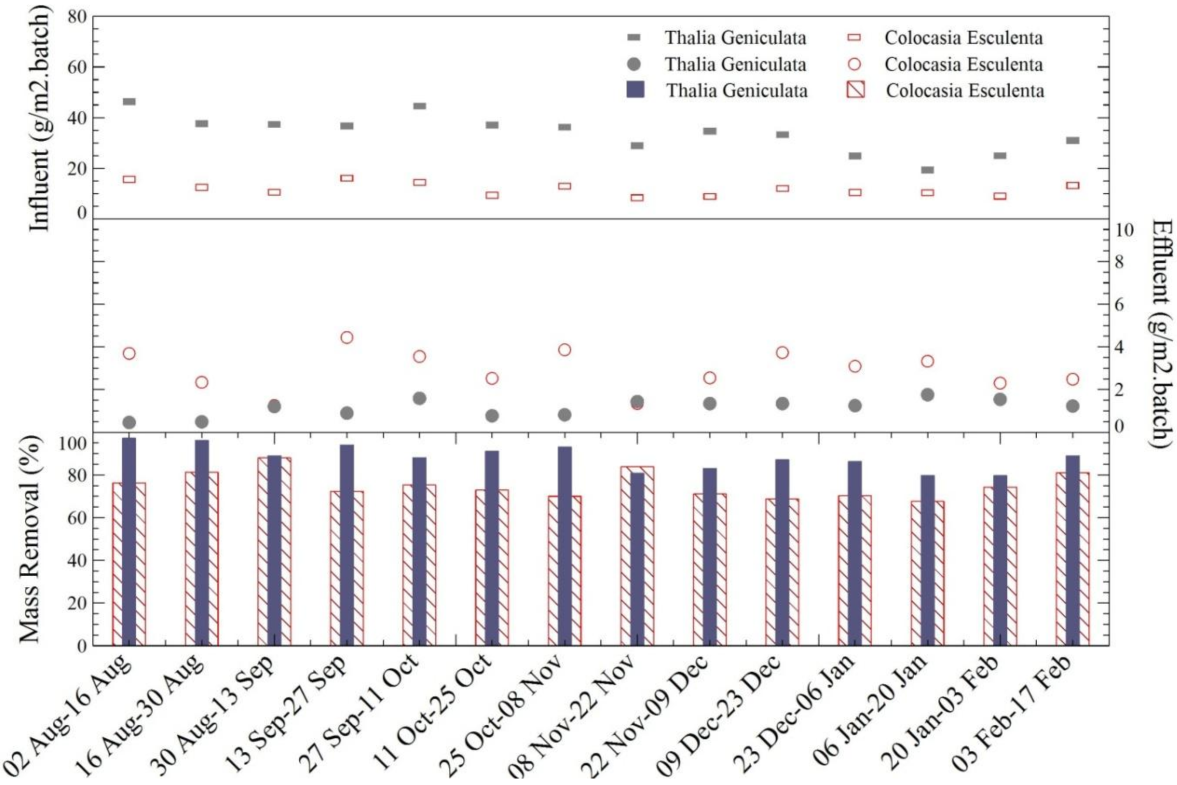


Figure 4.18 Influent and effluent TSS areal mass loading and mass removal efficiencies of the *Thalia geniculata* and *Colocasia esculenta* cells

There was a statistically significant difference between the mass removal efficiency of the four cells as determined by one-way ANOVA (p = *0.000*). Tukey post hoc test revealed that the *Typha Latifolia* cell statistically significantly outperformed the control cell (p = *0.008*) and the *Colocasia esculenta* cell (p = *0.024*). The *Thalia geniculata* also significantly outperformed the control cell (p = *0.000*) and the *Colocasia esculenta* cell (p = *0.000*). There were no statistically significant difference between the *Typha latifolia* cell and the *Thalia geniculata* cell (p = *0.155*) and also the control cell and *Colocasia esculenta* cell (p = *0.976*). The high mean TSS mass removal efficiencies indicated high solids retention capacity of both the planted and unplanted cells.

The higher removal efficiency of the *Colocasia esculenta* and *Thalia geniculata* cells highlights the importance of plants in TSS removal, which is similar to the submission of Gersberg *et al.,* (1986) that macrophyte root and roots and rhizomes, in addition to other physical processes that occur in the CW bed such as sedimentation and filtration, are largely responsible for the removal of TSS. However, the comparable performance of the control and *Colocasia esculenta* cells confirms the fact that the removal of TSS, which is primarily a physical process of settling and retention, may not be significantly affected by the presence or absence of macrophytes in a constructed wetland. Mburu *et al.,* (2008) concluded that filtration was the main mechanism responsible for the removal of suspended solids. Therefore, the higher removal efficiencies obtained for the *Typha latifolia* and *Thalia geniculata* cells confirms the occurrence of both biological and physical processes in the CW cells. There was a strong relationship between the loading rates and the removal rates in all the cells. Regression coefficient (R^2) ranged from 0.83 for the control cell to 0.98 for the *Thalia geniculata* cell, indicating a high predictability of the beds TSS mass removal rate. The observed relationship between influent loading rate and mass removal rate also corresponds to first-order kinetics.

4.2.4.4 NH_4-N and NO_3-N Removal

NH_4-N and NO_3-N effluent concentration ranged from 48.8 - 72.4mg/1 and 12 - 30mg/1 respectively for control cell; 52.9 - 120.9mg/1 and 17 - 69mg/1 respectively for the *Thalia geniculata* cell; 51.6 - 90.2mg/1 and 21 - 33mg/1 respectively for the *Colocasia esculenta* cell; and 48.8 - 92.1mg/1 and 16 - 47mg/1 respectively for the *Typha latifolia* cell. Effluent concentrations exceeded the influent concentrations in most cases as a result of water lost to ET. All values obtained exceeded the values 10 mg/l for NH_4-N and 20mg/l for NO_3-N prescribed by FEPA, (1991) for effluent discharge in Nigeria. Table 4.7 presents the NH_4-N influent and effluent areal loading rates.

Table 4.7 NH_4^+-N mass statistics for influent and effluent of the pilot wetland cells

		NH_4-N					
	Mass Loading (g/m^2.batch)	Min	Max	Mean	SD	MRR	RE (%)
Control	Influent	2.70	5.32	4.27	0.74		
	Effluent	2.06	3.40	2.97	0.38	1.30	29.89
Thalia geniculata	Influent	1.62	4.48	2.93	0.69		
	Effluent	0.75	2.60	1.72	0.45	1.14	41.33
Typha latifolia	Influent	1.81	4.75	3.23	0.71		
	Effluent	1.06	2.66	1.91	0.39	1.32	40.52
Colocasia esculenta	Influent	1.91	4.26	3.14	0.59		
	Effluent	1.06	2.52	1.91	0.33	1.22	38.79

MRR is the Mass removal rate and RE is the Removal efficiency

The influent NH_4-N mass loading in for the control cell ranged between 2.70 - 5.32g/m^2.batch with a mean value of 4.27 ± 0.74g/m^2.batch and an average mass removal rate (MRR) of 1.30kg/m^2. The influent NH_4-N mass loading in for the *Thalia geniculata* cell ranged between

1.62 - 4.48g/m^2.batch with a mean value of 2.93 ± 0.69g/m^2.batch and an average mass removal rate of 1.14kg/m^2.batch. The influent NH_4-N mass loading in for the *Typha latifolia* cell ranged between 1.81 - 4.75g/m^2.batch with a mean value of 3.23 ± 0.71g/m^2.batch and a mean mass removal rate of 1.32kg/m^2.batch. The influent NH_4-N mass loading in for the *Colocasia esculenta* cell ranged between 1.91 - 4.26g/m^2.batch with a mean value of 3.14 ± 0.59g/m^2.batch a mean mass removal rate of 1.22g/m^2.batch.

Figure 4.19 and 4.20 presents the influent and effluent NH_4-N areal loading and mass removal efficiencies for the different cells during the experimental run. The average NH_4-N mass removal efficiency for the control, *Thalia geniculata, Typha latifolia* and *Colocasia esculenta* cells were 29.89%, 41.33%, 40.51% and 38.79% respectively. Similar efficiency of 30% NH_4-N removal was recorded by Pucci *et al*. (2000), Ran *et al*. (2004) reported NH_4-N removal of 14%, while Kaseva (2003) recorded an NH_4-N removal efficiency of 11.2% - 25.2%. However, higher removal percentage of 54% was reported by Kadlec and Knight (1996) in their evaluation of the average performance of 70 North American wetlands treating domestic and agricultural effluent. Also Steer *et al.,* (2005) in their study of the performance of 8 subsurface horizontal flow constructed wetlands treating domestic effluent reported an average NH_4-N reduction of 70%.

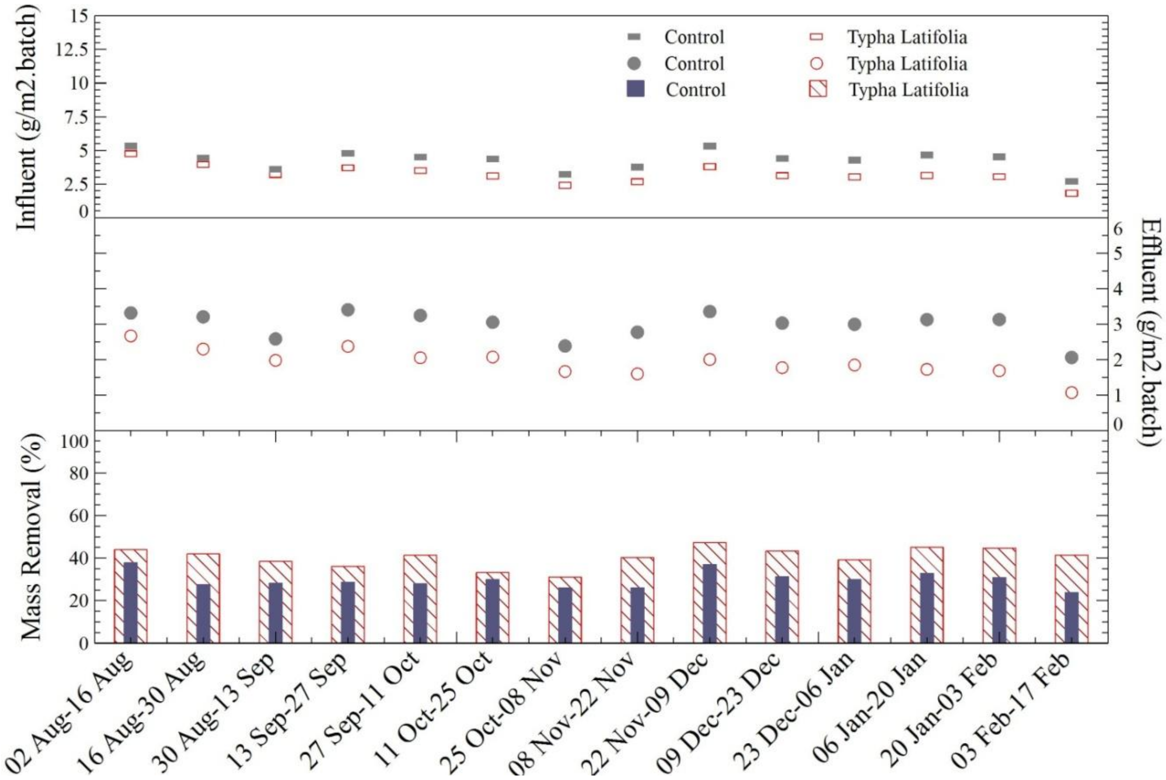


Figure 4.19 Influent and effluent NH_4-N areal mass loading and mass removal efficiencies of the control and *Typha latifolia* cells

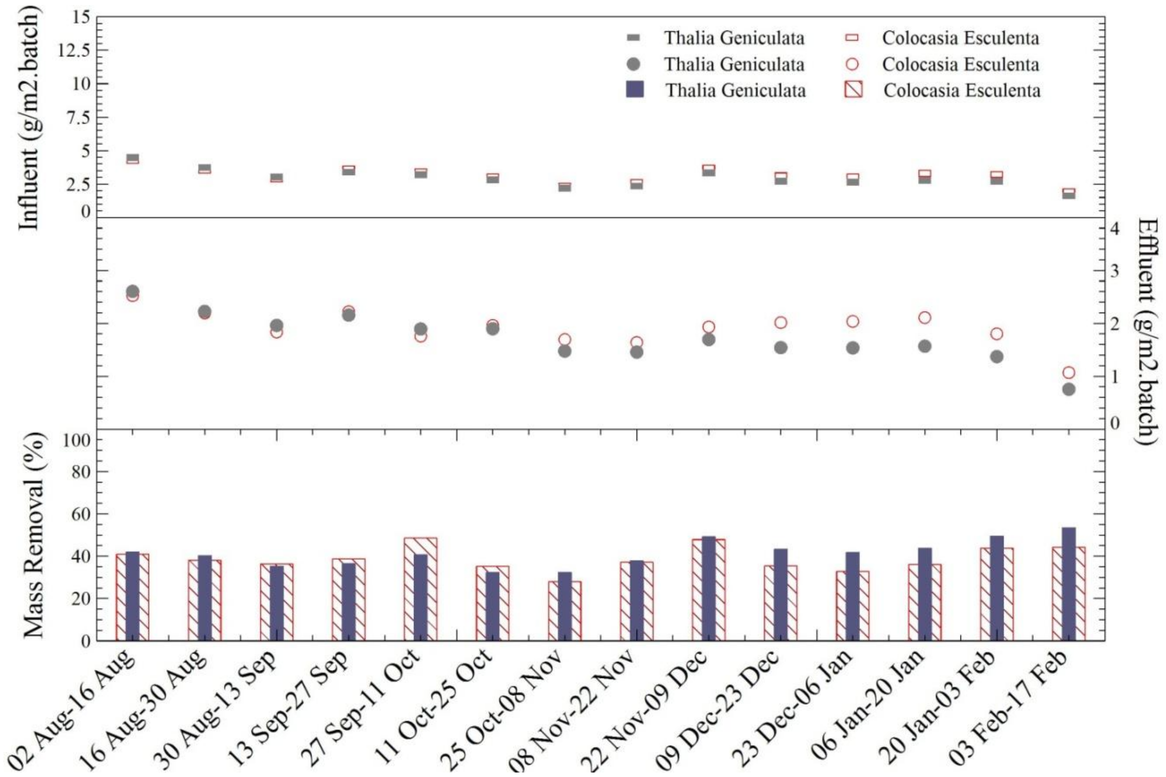


Figure 4.20 Influent and effluent NH_4-N areal mass loading and mass removal efficiencies of the *Thalia geniculata* and *Colocasia esculenta* cells

The variations in the removal rates recorded in the literature can be attributed to factors such as the differences in system design, with some systems consisting of a combination of surface flow and subsurface flow wetlands. Also variations in the retention times and plant species can also facilitate nitrification because some field scale wetlands that have been studied in the literature had very long retention times of up to 100days and were mixed species systems, which is known to maximize root biomass in the wetland substrate and results to higher aerobic degradation around the root zone (Brix, 1997).

There was a statistically significant difference between the mass removal efficiency of the four cells as determined by one-way ANOVA ($p = 0.000$). A Tukey post hoc test revealed that the unplanted cell was statistically significantly outperformed by the planted cells ($p = 0.000$ for *Thalia geniculata;* $p = 0.000$ for *Typha latifolia*; $p = 0.01$ for *Colocasia esculenta*). The mean difference in the removal efficiency between the cells with macrophyte and the control was 10.3%. However, no significant difference was obtained for *Typha latifolia* and *Thalia geniculata* cells ($p = 0.976$), *Typha latifolia* and *Colocasia esculenta* cells ($p = 0.714$), as well as *Thalia geniculata* and *Colocasia esculenta* cells ($p = 0.464$). The higher NH_4-N removal efficiencies achieved by the planted cells suggested that plant play an important role in NH_4-N removal. Besides direct uptake, their dense root and rhizomes system provides large surface area for attachment of microorganisms conducive for microbial metabolic activities, besides transporting atmospheric oxygen into the substrate through the plants tissues. The root mat allows for enhanced removal of particulates that are trapped in the bed. George *et al.,* (2000) also observed better NH_4-N removal in pilot scale subsurface flow wetlands planted with bulrush than beds without plants.

The overall low removal rates achieved by the wetlands in this study can be attributed to a lot of factors. The main mechanism of NH_4-N removal from wastewater has been identified by many authors to be the microbial mediated sequential nitrification - denitrification: the aerobic oxidation of ammonium to nitrite by ammonium oxidizing bacteria and the subsequent oxidation of the produced nitrite to nitrate by nitrite oxidizing bacteria (USEPA, 1999, Vymazal *et al*, 2007). The extent to which this processes can progress is governed by the availability of oxygen (Kadlec and Knight, 1996). Sources of Oxygen in CWs include dissolved oxygen in the wastewater, by surface aeration, oxygen translocation to the rhizosphere by plants and photosynthetic generation by phytoplankton (Liu *et al*., 2016; Rehman *et al*., 2016; Zhang *et al*., 2014).

Agricultural wastewater is known to be very low on dissolved oxygen (Poach *et al*., 2003). The capacity of different macrophytes species to traslocate oxygen and the quantity of oxygen transferable by macrophytes in constructed wetlands are still issues of considerable discussion. Suwasa *et al.*, (2008) demonstrated that the surface area must be large enough to secure a sufficient oxygen transfer to cover the need for microbial degradation of organic matter and nitrification of ammonium. The oxygen supply to the rhizosphere over that duration of this experimental study was most probably restricted to the amount translocated by the shoots, since surface aeration and photosynthetic transfer were limited by the intense plant cover. However, it has been established that the transfered from the macrophyte shoots to the root mat is barely enough for their respiratory need and microorganism only compete for what is left after plant respiration. Therefore, other mechanisms such as adsorption and plant uptake may likely have played very significant roles in the observed nitrogen removal.

Furthermore, the short establishment period prior to experimental treatments may have also accounted for the low removal rate due to poor root mat development, which can decrease the aerobic regions in the constructed wetland. According to Kadlec *et al*. (2000), it takes about 3 to 5 years for the root and rhizome of macrophytes to develope completely. Wood (1990) also suggested six to twelve months for vegetation to adequately develop active rhizosphere. Also, NH_4-N removal in CWs is influenced by the residence time of wastewater in the system. Longer HRT is important because for nitrogenous BOD to be reduced, carbonaceous BOD must be reduced first to a relatively low concentration (< 40 mg/L), as carbonaceous BOD inhibits the activity of nitrifying bacteria (Kadlec and Knight, 1996).

Table 4.8 presents the NO_3-N influent and effluent areal loading rates.

Table 4.8 NO_3^--N mass statistics for influent and effluent of the pilot wetland cells

		NO_3-N					
	Mass Loading (g/m^2.batch)	Min	Max	Mean	SD	MRR	RE (%)
Control	Influent	1.69	3.21	2.30	0.47		
	Effluent	0.63	1.38	0.98	0.23	1.31	56.52
Thalia geniculata	Influent	1.08	2.26	1.56	0.36		
	Effluent	0.47	1.13	0.68	0.21	0.84	55.86
Typha latifolia	Influent	1.20	2.47	1.73	0.37		
	Effluent	0.61	1.35	0.76	0.22	0.96	55.32
Colocasia esculenta	Influent	1.20	2.41	1.69	0.35		
	Effluent	0.56	0.89	0.71	0.10	0.97	56.58

Note: MRR is the Mass removal rate and RE is the Removal efficiency

The influent NO_3-N mass loading for the control cell ranged between 1.69 - 3.21g/m^2.batch with a mean value of 2.30 ± 0.47g/m^2.batch and a mean batch mass removal rate of 1.13g/m^2.batch. The influent NO_3-N mass loading for the *Thalia geniculata* cell ranged between 1.08 - 2.26g/m^2.batch with a mean value of 1.56 ± 0.36g/m^2.batch and a mean mass removal rate of 0.84 g/m^2.batch. The influent NO_3-N mass loading for the *Typha latifolia* cell ranged between

1.20 - 2.47g/m^2.batch with a mean value of 1.73 ± 0.37g/m^2.batch and a mean mass removal rate of 0.96g/m^2.batch. The influent NO_3-N mass loading for the *Colocasia esculenta* cell ranged between 1.20 - 2.41g/m^2.batch with a mean value of 1.69 ± 0.35g/m^2.batch and an average mass removal rate of 0.97g/m^2.batch.

Figure 4.21 and 4.22 presents the influent and effluent NO_3-N areal loading and mass removal efficiencies for the different cells during the experimental run. The average NO_3-N mass removal efficiency were 56.5%, 55.8%, 55.3% and 56.6% for control, *Thalia geniculata*, *Typha latifolia* and *Colocasia esculenta* cells respectively. The NO_3-N removal rates achieved by the cells were consistent with ranges reported in the literature such as 61.0% and 65.0% reported by Badejo *et al.,* (2012) for wetlands with *Vetiveria nigritana and Phragmites karka* respectively treating tertiary hospital wastewater in Nigeria. Kadlec and Knight (1996) reported average values of 61% for NO_3-N removal in Europe, while Kovacic *et al.,* (2000) in their examination of NO_3-N removal from constructed wetlands receiving agricultural tile drainage reported a removal percentage of 36%. Also Kassa and Mengistou (2014) in their study of nutrient uptake efficiency and growth of two aquatic macrophyte species in constructed wetlands in Ethiopia reported a mean removal efficiency of 56.37% for the treatment bed planted with *Cyperus papyrus*. Higher removal efficiency of 74.62% was reported by Dhulap and Patil (2014) for sewage treatment using CW.

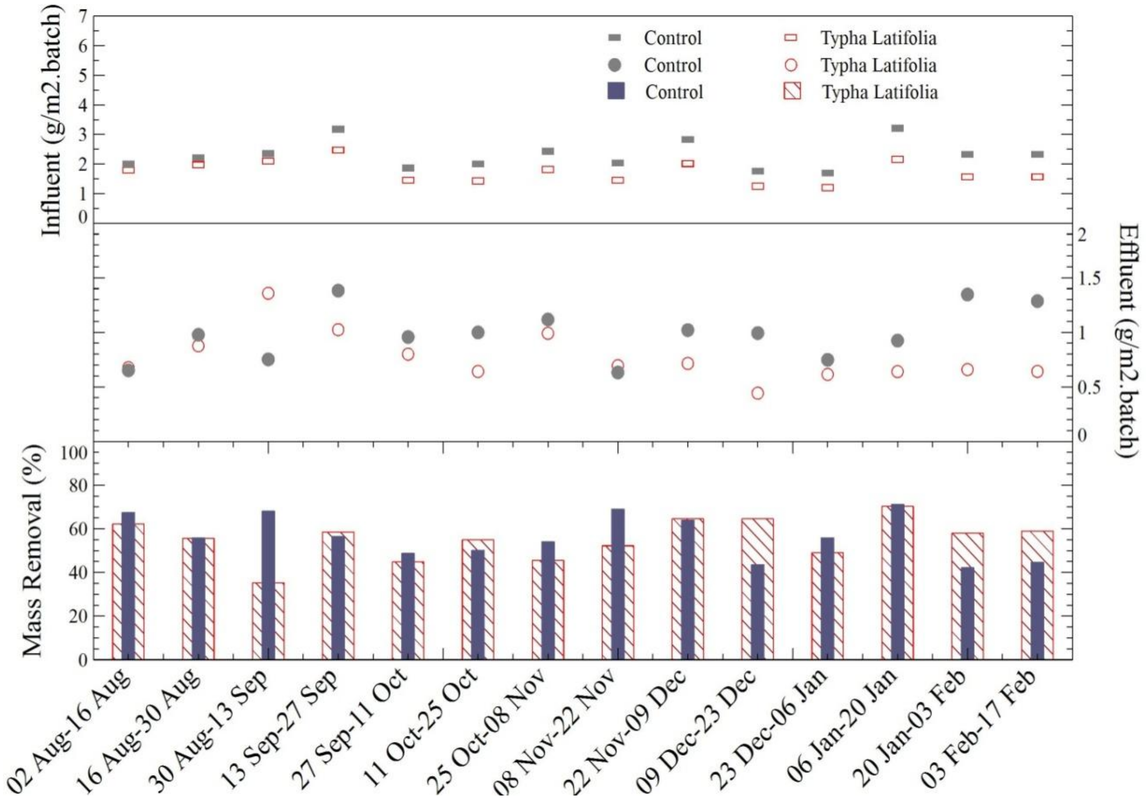


Figure 4.21 Influent and effluent NO_3-N areal mass loading and mass removal efficiencies of the control and *Typha Latifolia* cells

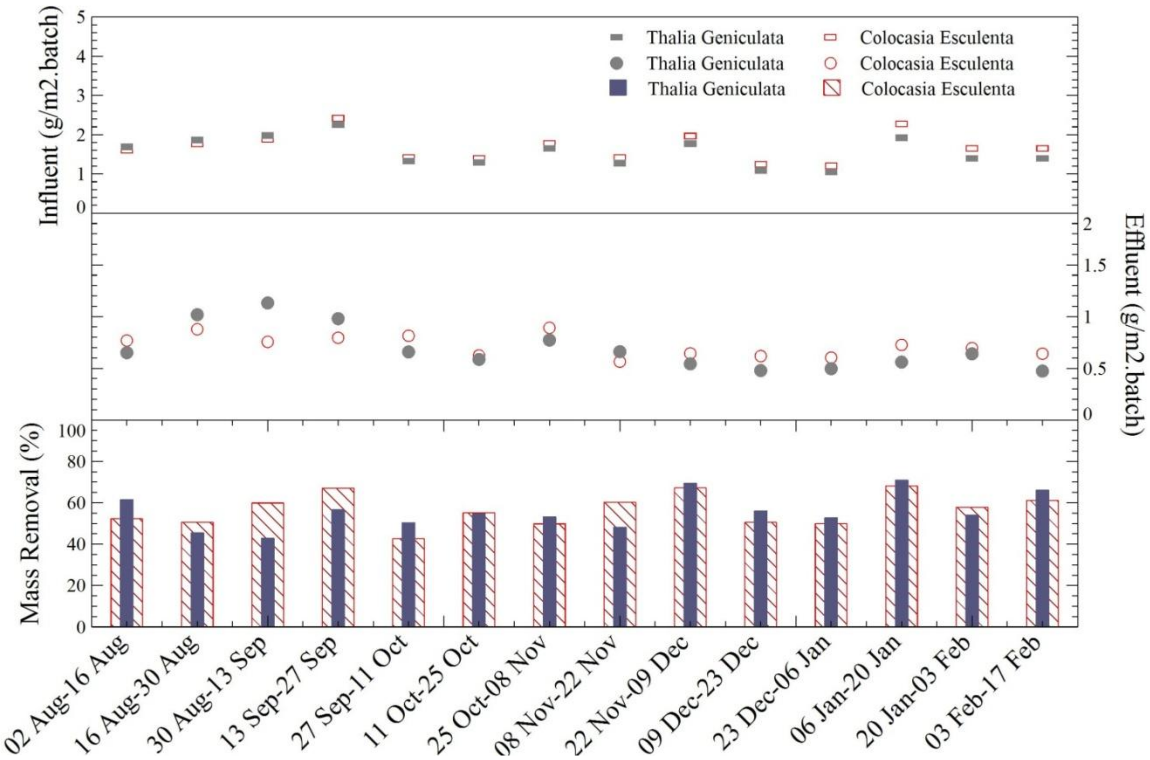


Figure 4.22 Influent and effluent NO_3-N areal mass loading and mass removal efficiencies of the *Thalia geniculata* and *Colocasia esculenta* cells

The mass removal efficiency of the four cells as determined by one-way ANOVA showed no statistically significant difference ($p = 0.979$). Several experimental studies on NO_3-N removal in constructed wetlands have confirmed that unplanted treatment had lower nitrogen removal compared with planted treatment (Yang *et al*., 2001; Tadesse 2010; Lin *et al*., 2002). In this study, the lower mean NO_3-N removal in the planted beds may be due to higher nitrification rate in the planted beds due to higher oxygen translocation from the root–rhizome system, as studies have shown that the predominant removal mechanism of NO_3-N from wastewater was by denitrification (Xue *et al*., 1999). Thus higher dissolved oxygen in planted beds imposed a restriction on denitrification, while aiding the conversion of ammonium nitrogen to nitrate nitrogen. Plant uptake did not play a significant role as have been reported in literature (Vymazal, 2007) and is known to constitute 2 - 10 % of nitrogen removal (Tanner, 1996). However, the very low plant uptake, evident by the fact that the unplanted bed outperformed some of the planted beds may be attributed to the root-rhizome development.

The influence of NH_4-N and NO_3-N loading on their removal rates in the cells were similar to the observed relationship for BOD and TSS. Linear relationships existed for both the NH_4-N and NO_3-N. R^2 ranged from 0.69 for the *Thalia Geniculata* cell to 0.91 for the control cell for NH_4-N, and 0.63 for the *Typha Latifolia* cell to 0.92 for the *Colocasia Esculenta* cell for NO_3-N. The linearity exhibited suggests that the wetland could be operated at elevated hydraulic loads. The observed relationship between influent loading rate and mass removal rate also also followed first-order kinetics.

4.2.4.4 PO_4^{3-} Removal

PO_4^{3-} effluent concentration ranged from 3 - 14mg/1 for control cell; 2 - 33mg/1 for the *Thalia geniculata* cell; 2 - 14 mg/1 for the *Colocasia esculenta* cell; and 2 - 16mg/1 for the *Typha latifolia* cell. The higher effluent concentrations obtained in some cases, compared to the influent concentrations, were mainly as a result of the water lost to ET which translated to increased pollutant concentrations.

Table 4.9 summarizes the PO_4^{3-} influent and effluent areal loading rates. The influent PO_4^{3-} mass loading for the control cell ranged between 0.32 - 0.82g/m^2.batch with a mean value of 0.61 ± 0.16g/m^2.batch and an average mass removal rate (MRR) of 0.25g/m^2.batch. The influent PO_4^{3-} mass loading for the *Thalia geniculata* cell ranged between 0.21 - 0.70g/m^2.batch with a mean value of 0.42 ± 0.13g/m^2.batch and an average mass removal rate of 0.21g/m^2.batch. The influent PO_4^{3-} mass loading for the *Typha latifolia* cell ranged between 0.23 - 0.74g/m^2.batch with a mean value of 0.46 ± 0.14g/m^2.batch and a mean mass removal rate of 0.25g/m^2.batch. The effluent PO_4^{3-} mass loading for the *Colocasia esculenta* cell ranged between 0.22 - 0.66g/m^2.batch with a mean value of 0.45 ± 0.12g/m^2.batch a mean mass removal rate of 0.28g/m^2.batch.

Table 4.9 PO_4^{3-} mass statistics for influent and effluent of the pilot wetland cells

		PO_4^{3-}					
	Mass Loading (g/m^2.batch)	Min	Max	Mean	SD	MRR	RE (%)
Control	Influent	0.32	0.82	0.61	0.16		
	Effluent	0.16	0.63	0.36	0.14	0.25	41.40
Thalia geniculata	Influent	0.21	0.70	0.42	0.13		
	Effluent	0.07	0.47	0.21	0.10	0.21	44.73
Typha latifolia	Influent	0.23	0.74	0.46	0.14		
	Effluent	0.08	0.42	0.21	0.10	0.25	52.09
Colocasia esculenta	Influent	0.22	0.66	0.45	0.12		
	Effluent	0.07	0.27	0.17	0.05	0.28	61.50

Note: MRR is the Mass removal rate and RE is the Removal efficiency

Figure 4.23 and 4.24 presents the influent and effluent PO_4^{3-} areal loading and mass removal efficiencies for the different cells during the experimental run.. The average PO_4^{3-} mass removal efficiency for the control, *Thalia geniculata*, *Typha latifolia* and *Colocasia esculenta* cells were 41.40%, 44.73%, 52.09% and 61.50% respectively. The performance achieved by the wetland cells in this pilot study was within the range reported for P removal as indicated by the results of Chang *et al.*, (2012), 51.1% to 52.0%; Dhulap and Patil (2014), 57.81% and Badejo *et al.*, (2012), 81.0%.

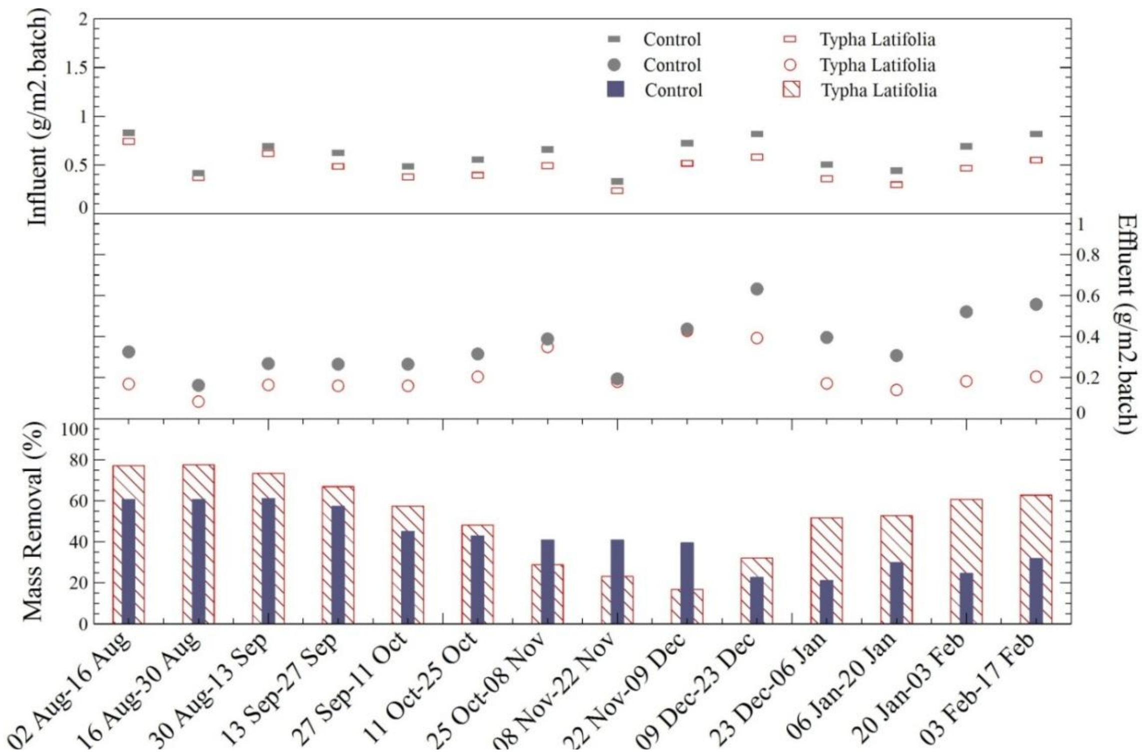


Figure 4.23 Influent and effluent PO_4^{3-} areal mass loading and mass removal efficiencies of the Control and *Typha latifolia* cells

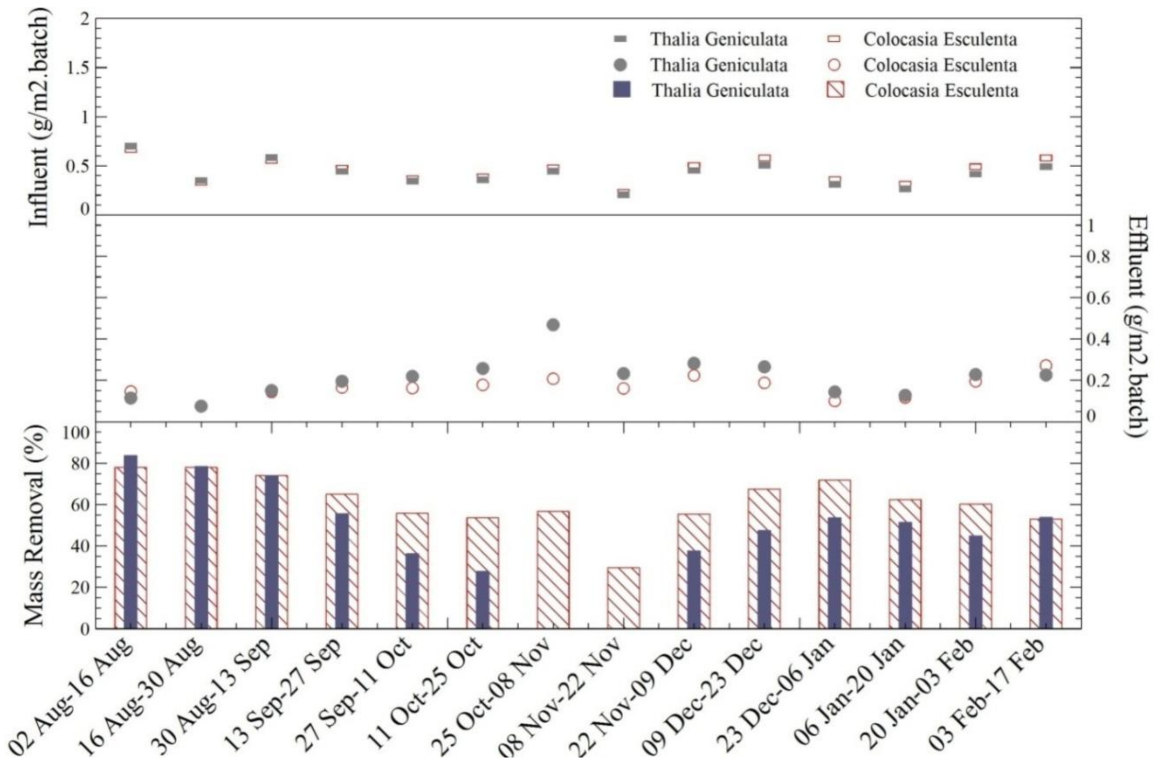


Figure 4.24 Influent and effluent PO_4^{3-}areal mass loading and mass removal efficiencies of the *Thalia geniculata* and *Colocasia esculenta* cells

There was a statistically significant difference ($p = 0.045$) between the mass removal efficiency of the four cells as determined by one-way ANOVA. A Tukey post hoc test revealed that the *Colocasia esculenta* cell significantly ($p = 0.044$) outperformed the unplanted cells. However, there was no statistically significant difference ($p > 0.05$) between the control and the other planted cells. Also there was no statistically significant difference ($p > 0.05$) amongst the planted cells. Kadlec and Knight (1996) argued that biogeochemical processes control phosphorus removal from wastewater by wetlands. Adsorption to the soil or substrata is proposed as the main mechanism that contributes to a large part of the removal (Wood, 1990, Vymazal, 2007). This was the probable reason for the insignificant difference between the unplanted cell and the cells planted with *Thalia geniculata* and *Typha latifolia*. Other routes such as phytoplankton intake, other algae and plants are recognized as offering only temporary phosphorus storage. Reduction of PO_4^{3-} obtained from mass budgets in all studied wetland cells was characterized by initial increased removal rates and a subsequent decrease over time. The decrease in the *Thalia geniculata* cell eventually led to a release (minimum value of -13%). This trend suggested saturation of removal routes within the wetland beds that include adsorption through the substrate material. The reduced phosphate reduction over time can also be associated with the release of stored phosphorus through the decaying litter.

Macrophytes create a hospitable habitat for many decomposing microorganisms in the rhizosphere and play an indirect but important role in the reduction of organic matter and nitrogen from various types of wastewater. The results of this research have supported the fact that macrophytes are a vital part of the treatment system. However, mechanisms involved in phytoremediation in CWs are still being discussed. This is particularly evident in the findings of this study which have shown that some macrophytes perform better for organic substances and are poor in removing nutrients. Shele *et al*., (2013) stated that contradictory findings have been reported from different experimental strategies with different plants. These contradictions increases when comparison are made betweeb wetlands of different types. Literature finding have shown that polyculture are more effective in pollutant removal than monoculture. Karathanasis *et al*., (2003) argued that polyculture systems seemed to offer the best and most consistent treatment for all wastewater parameters, while being least susceptible to seasonal variations, and concluded that the presence of various species may have provided a more effective distribution of the rooting biomass and habitat for more diverse microbial populations than the monoculture systems. Therefore, a combination of the studied macrophytes is recommended.

4.2.5 Wastewater Treatment Performance of PKS in a Pilot Wetland

Table 4.10 shows the pollutant mass statistics in terms of influent and effluent areal loading rates for the PKS and gravel wetland cells. The mean influent BOD mass loading for the PKS cell was 27.74 ± 6.45g/m^2.batch and a mean mass removal rates (MRRs) of 20.02g/m^2.batch, against a mean MMR of 25.15g/m^2.batch obtained for the gravel bed. The mean influent TSS mass loading for the PKS cell was 8.92 ± 2.43g/m^2.batch and a mean MRR of 7.98g/m^2.batch, against a mean MMR of 9.53g/m^2.batch obtained for the gravel bed. The mean influent NH_4-N mass loading for the PKS cell was 2.41 ± 0.61g/m^2.batch and a mean MRR of 0.88g/m^2.batch, against a mean MMR of 1.13g/m^2.batch obtained for the gravel bed. The mean influent NO_3-N mass loading for the PKS cell was 1.29 ± 0.31g/m^2.batch and a mean MRR of 0.76g/m^2.batch, against a mean MMR of 0.84g/m^2.batch obtained for the gravel bed. The mean influent PO_4^{3-} mass loading for the PKS cell was 0.34 ± 0.11 g/m^2.batch and a mean MRR of 0.15 g/m^2.batch, against a mean MMR of 0.21 g/m^2.batch obtained for the gravel bed.

Table 4.10 Pollutants mass statistics for influent and effluent of PKS and gravel wetland cells

		BOD			TSS			NH_4-N		
	Mass (g/m^2.batch)	Mean±SD	MRR	RE (%)	Mean±SD	MRR	RE (%)	Mean±SD	MRR	RE (%)
PKS	Influent	27.74±6.45			8.92±2.43			2.41±0.61		
	Effluent	7.38±2.16	20.02	72.81	0.82±0.30	7.98	89.87	1.47±0.44	0.88	39.42
Gravel	Influent	33.75±7.48			10.85±2.83			2.93±0.70		
	Effluent	8.07±2.60	25.15	75.42	1.15±0.40	9.53	88.18	1.72±0.44	1.13	41.33

		NO_3-N			PO_4^{3-}		
	Mass (g/m^2.batch)	Mean±SD	MRR	RE (%)	Mean±SD	MRR	RE (%)
PKS	Influent	1.29±0.31			0.34±0.11		
	Effluent	0.50±0.15	0.76	60.79	0.19±0.05	0.15	42.52
Gravel	Influent	1.57±0.36			0.42±0.13		
	Effluent	0.69±0.21	0.84	55.86	0.21±0.10	0.21	44.73

Figure 4.25 to 4.29 shows the pollutant mass removal efficiencies for the PKS and gravel cells. The average BOD mass removal efficiency for the PKS and gravel cells were 72.81% and 75.42% respectively. Average TSS mass removal efficiency for the PKS and gravel cells were 89.87% and 88.18% respectively. Average NH_4-N mass removal efficiency for the PKS and gravel cells were 39.42% and 41.33% respectively. Average NO_3-N mass removal efficiency for the PKS and gravel cells were 60.79% and 55.86% respectively. Average PO_4^{3-} mass removal efficiency for the PKS and gravel cells were 42.52% and 44.73% respectively.

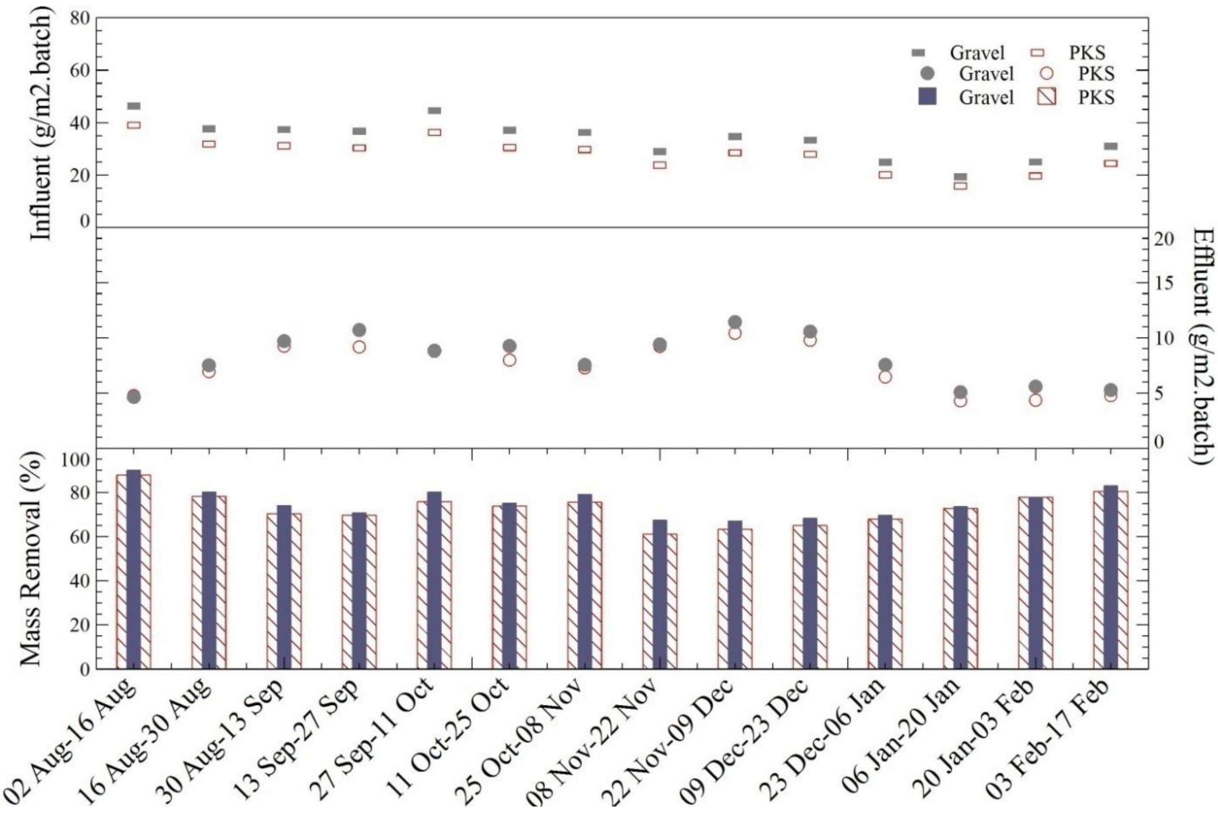


Figure 4.25 Influent and effluent BOD mass flux and removal efficiencies for the PKS and gravel cells.

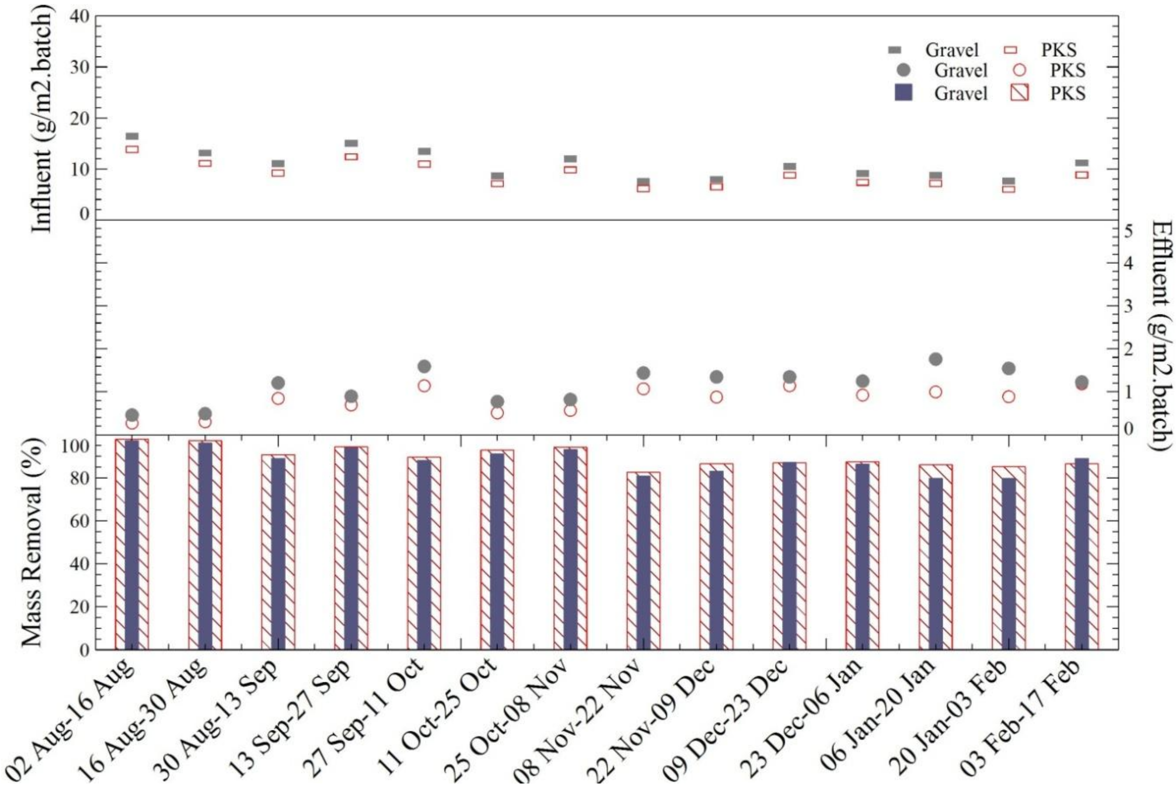


Figure 4.26 Influent and effluent TSS mass flux and removal efficiencies for the PKS and gravel cells.

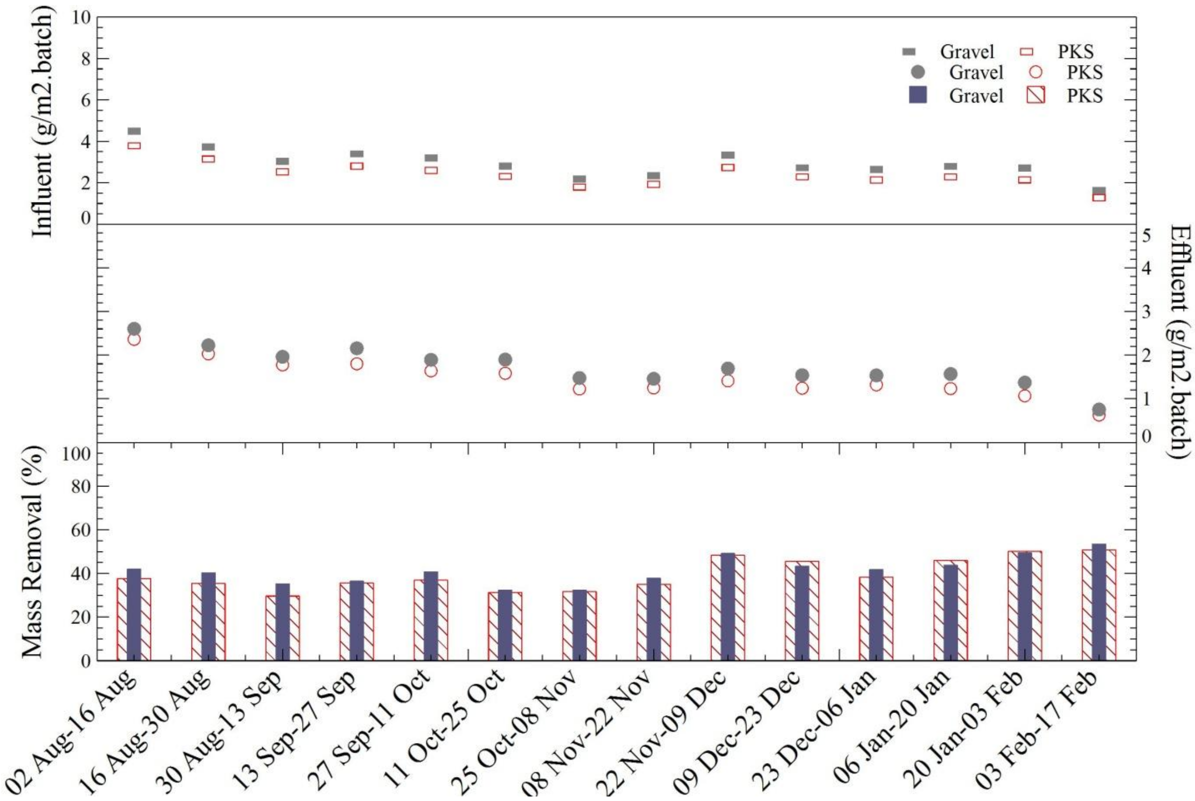


Figure 4.27 Influent and effluent NH_4-N mass flux and removal efficiencies for the PKS and gravel cells.

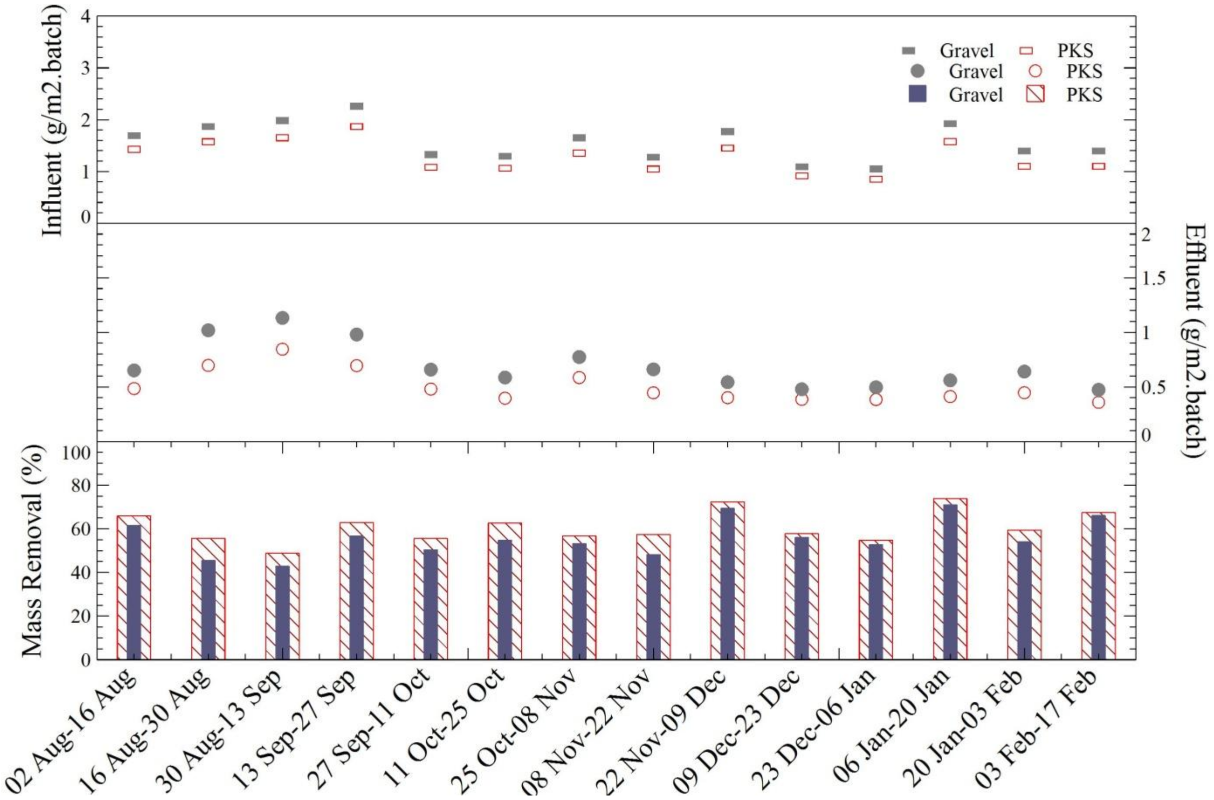


Figure 4.28 Influent and effluent NO_3-N mass flux and removal efficiencies for the PKS and gravel cells.

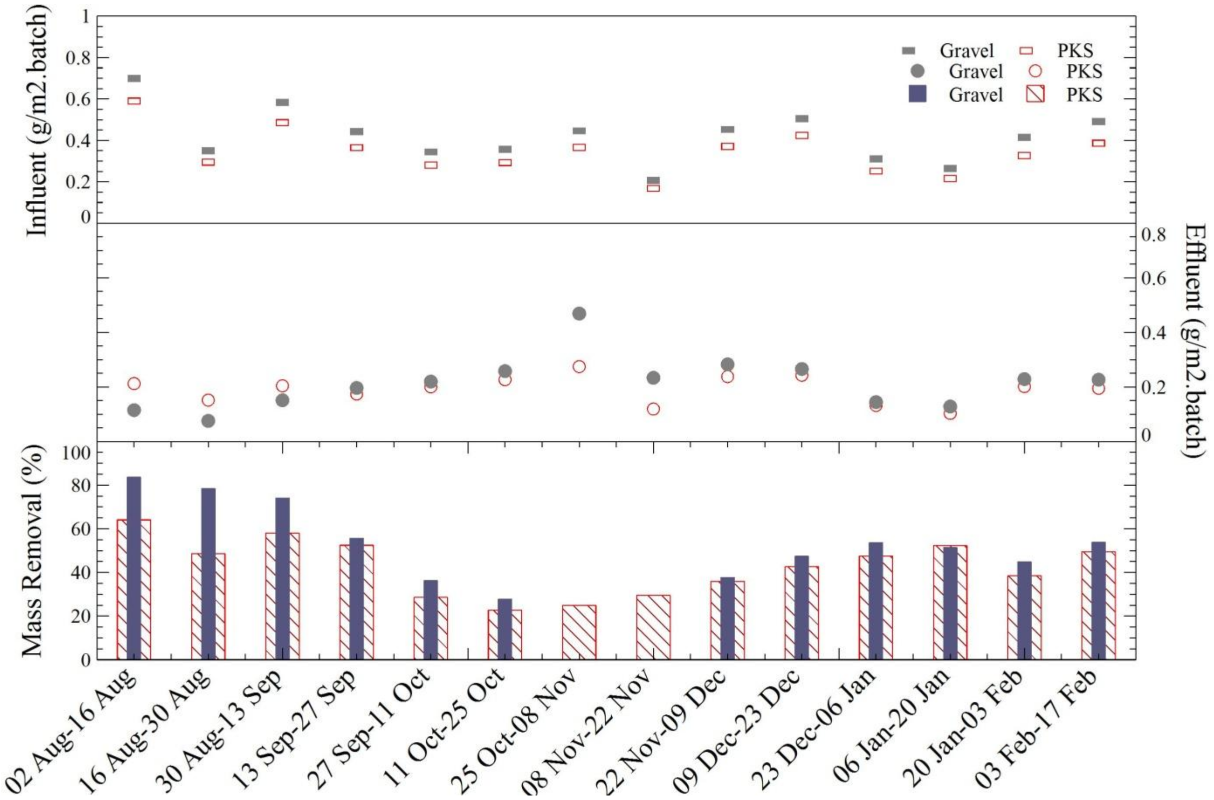


Figure 4.39 Influent and effluent PO_4^{3-}mass flux and removal efficiencies for the PKS and gravel cells.

The performance of the cells in terms of organic matter removal was satisfactory with more than 70% reduction in both PKS and gravel beds. Although the gravel bed had a higher mean BOD removal efficiency than the PKS bed, the difference between them was not statistically significant ($p = 0.331$). The PKS media being an organic substrate could potentially release soluble organic matter into the bed effluent, thereby increasing the BOD content in the outflow water, leading to a higher value than the gravel bed. The slightly lower removal rate obtained for the PKS bed can also be attributed to a lower DO concentration as presence of PKS results to an additional oxygen consumption which creates more anaerobic microsites that may inhibit the biodegradation of carbonaceous compound by aerobic microorganisms.

The mean percentage TSS mass removal of the PKS and gravel beds were not significantly different ($p = 0.410$). Both beds demonstrated comparable filterabilty, attaining similarly high removal of suspended solids irrespective of the substrate material. The resulting effluent from the beds were evidently clearer and free of visible suspended matter upon exit from the wetlands, which may be attributed to the fact that TSS removal takes place through physical processes. Generally, higher NH_4-N removal efficiency was found in the gravel beds compared to the PKS bed. However, no statistically significant difference ($p = 0.465$) was found with the NH_4-N treatment efficiencies between the two beds. The higher mean mass removal rate obtained for the gravel bed can be attributed to the likely higher DO content in the bed. Nitrification is heavily dependent on the presence of DO. The condition in the PKS bed is likely to be more anaerobic than aerobic. PKS as an additional carbon source, which is known to contributed to greater growth and biomass of heterotrophs did not significantly influence the ammonia removal efficiencies, which can be attributed to the fact that ammonia oxidisers compete poorly with aerobic heterotrophic microorganisms (Vymazal, 2007).

Higher NO_3-N mass removal efficiency was obtained in the PKS bed compared to the gravel bed indicating the superiority of the PKS bed over the gravel bed in terms of NO_3-N reduction, although the result was not statistically significant (*p = 0.107*). According to Horne (1995), denitrification can be induced with oxygen levels less than 0.2 mg/L, a sufficient supply of nitrate and carbon food, and the presence of a physical site where the bacteria required in the process can attach to. PKS acted as additional carbon source to support denitrification and also, the expected lower DO concentration in the PKS bed as a results of additional oxygen consumption by the organic media, must have created more anaerobic microsites which is regarded as a determining factor for NO_3-N removal. Higher PO_4^{3-} mass removal efficiency was initially recorded for the gravel bed compared PKS bed. However, saturation of removal routes in the gravel bed, which led to PO_4^{3-} release, was not observed in the PKS bed, which indicated that its removal routes had not been saturated. Also the difference in the mean removal efficiencies of both systems were found not to be statistically significant (*p = 0.790*). The overall performance of the PKS bed as regards pollutant removal from slaughterhouse wastewater was satisfactory when compared to the gravel media. This is in line with the conclusions of Chong *et al*., (2009) that PKS performed better than the conventional constructed wetland medium and therefore it is a better medium for constructed wetland application.

The durability of an aggregate is a measure of its resistance to wear, moisture penetration, decay and disintergration (Olanipekun *et al*., 2006). The durability of the PKS in pilot HSSF CW was evaluated over 20 months operational period, by determining three main properties as shown in Table 4.11. The test results show that the specific gravity obtained for the PKS at the start of the experiment was 1.34 compared to the typical value of 2.65 for granite aggregate. After 6 month in the pilot HSSF CW, the specific gravity reduced slightly to 1.28. However, after 20 months in the system, there was a significant reduction in the specific gravity to a value of 0.96. Similar trend were observed for aggregate crushing values. The aggregate crushing value, which is the relative measure of the resistance of an aggregate to crushing under a gradually applied compressive load, was 5.3% at the start of the experiment in May 2016. The value obtained for the PKS was lower than the typical range of 24-26% for granite aggregates, which shows that palm kernel shell aggregates are stronger under loads than the normal weight aggregates. The aggregate crushing value of the PKS increased to 8.7% and 14.9% after 6 and 20 months in the HSSF CW.

Table 4.11 Physical and mechanical properties of the PKS shell

Properties	**May 2016**	**November 2016**	**January 2018**
Specific Gravity	1.34	1.28	0.96
Shell Thickness (mm)	4.70 ± 0.19	4.67 ± 0.17	4.64 ± 0.19
Aggregate Crushing Value (%)	5.3	8.7	14.9

There was no significant variation in the mean shell thickness over the 20 months of operation. The mean thickness of the selected shells at the start of the experiment was 4.70 mm and reduced slightly to 4.64 mm after 20 months in the HSSF CW. PKS has been used for different purposes in the construction industry because of its relative abundance and certain properties such as high compaction, low density and strong interlocking properties (Amu *et al*., 2008). Okoroigwe *et al*., (2014) stated that some characteristics of PKS support its application as both construction material filler and water treatment agent in the food and beverage industry. However, the results of this study have shown that PKS, being a biodegradable material is subject to considerable deterioration over time. Therefore, further studies to investigate the strength and durability of PKS as a constructed wetland substrate is recommended.

4.3 *k-C** Model Fitting and Verification

The main design consideration for constructed wetland is to find the correct dimensions of the system, so that the wastewater has enough time to stay in the system, to ensure that the parameter concentrations are below the statutory values. In order to use the first order kinetic model as a basis for design, the correct reaction rate constant is needed, which is influenced by environmental and ecosystem characteristics. In others to estimate the model constants for a PKS-based CW under tropical environmental conditions, the average data of the two planted HSSF CW column were used to calibrate the modified first order plug flow model of Kadlec and Knight (1996). Values of rate constant at 20 °C (k_{20}), temperature coefficient (θ) and background concentration (C^*) were calibrated simultaneously to produce concentration profiles that most closely match the field wastewater parameter concentrations.

Figures 4.30 and 4.31 respectively illustrate the mean observed and fitted BOD and TSS dynamics in the HSSF CW column. The input BOD and TSS concentration were 636.23mg/l and 483mg/l, respectively. In the figures, the X-axis represents the hydraulic retention time (HRT) in days and the Y-axis the concentration of the pollutant. As can be seen from the figure, the BOD reduction followed exponential trends and the modified first order kinetic model with a rate constant (k_{20}) of 0.604/day (32.63m/year), temperature coefficient (θ) of 0.995 and a background concentration (C^*) of 43.23mg/l matched the data of the experimental run (coefficient of determination for regression R^2 = 0.897). BOD decreased rapidly in the first five days of incubation and approached residual levels (C^*) on day 12. Also the TSS reduction followed exponential trends and the modified first order kinetic model with a rate constant (k_{20}) of 0.623/day (33.65m/y), temperature coefficient (θ) of 1.093 and a background concentration (C^*) of 33.77mg/l matched the data of the experimental run (R^2 = 0.525).

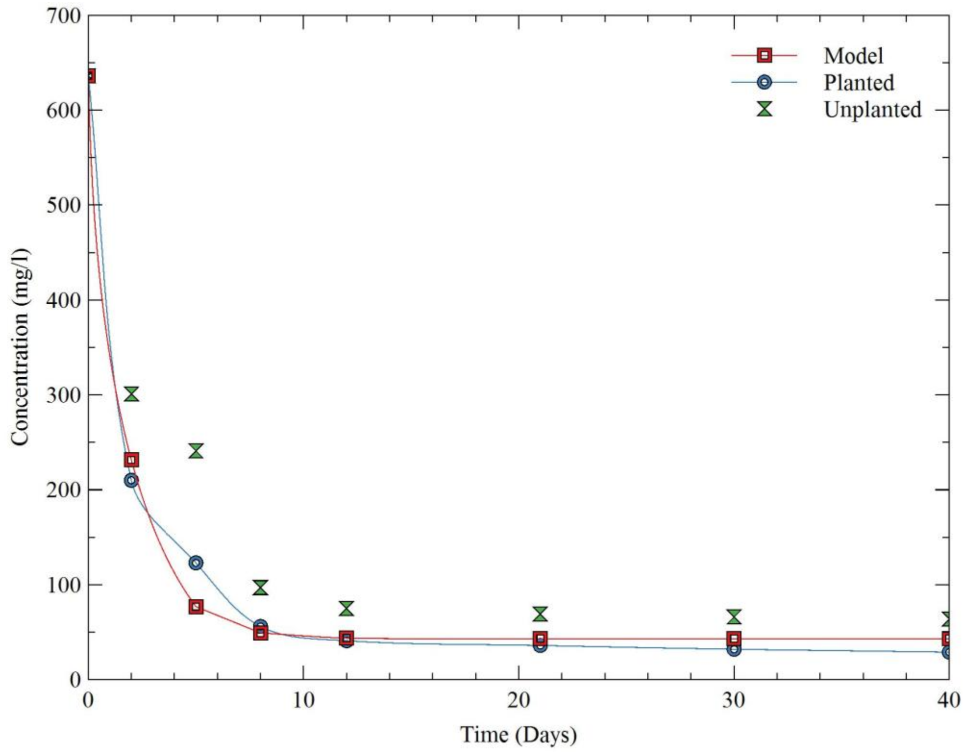


Figure 4.30 Measured and fitted BOD dynamics during batch incubations in the HSSF-CW column.

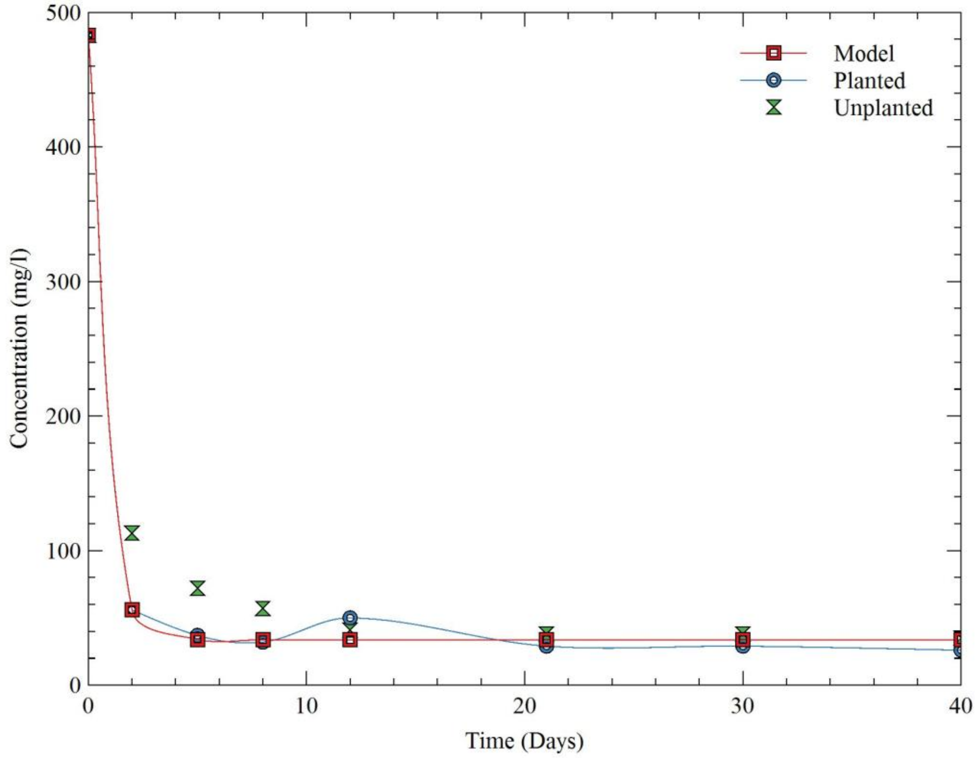


Figure 4.31 Measured and fitted TSS dynamics during batch incubations in the HSSF-CW column.

There are no removal rates reported in literature for BOD and TSS in CWs using PKS as media. However, the rate constant of 0.604/day (32.63m/yr) obtained for BOD in this study was within ranges of 0.17/day to 6.11/day previously reported for different types of CW (0.17/day by Tanner *et al*., 1995; 0.3 - 6.11 /day by Kadlec and Knight, 1996; 0.87 /day by Lin et al., 2002; 1.104/day by Reed and Brown, 1995; 0.86 /day by Liu *et al*., 2000) and the rate constant of 0.623/day (33.65m/yr) for TSS also falls within the ranges of 0.27/day to 4.11/day previously reported (0.20/day by Cosmos, 2006; 4.11 /day by Wong *et al*., 2006). The temperature coefficient (θ) describes the temperature dependency and a value of 1.000 indicates that the temperature does not influence the treatment, while values below or above 1.000 have a negative or positive effect on the treatment (Kadlec and Wallace, 2009). The estimated values for the temperature coefficient for BOD and TSS were comparable to those reported by Kadlec and Knight (1996). The slightly less than unit value obtained for BOD suggested a slightly lower removal rate at higher temperatures. This is not consistent with concepts of microbial degradation and must therefore be viewed with skepticism. Possible explanations are the relatively short duration of the column experiments, which may have occurred in a grow-in period for plants and microbes.

The values of BOD background concentration (C^*) obtained in this study were higher than the values recommended by Kadlec and Knight (1996) for system design. Possible explanations are the very short duration of column experiments. In the literature, a range of $1.7 < C^* \leq 18.2$mg/l with an average of 9.9 mg/l was reported by Stein *et al*., (2006). The lower values in the literature can be attributed to the fact that their values were long-term averages of residual values obtained from different systems and it is a known fact that the values of k and C^* vary from one wetland to another, depending on site-specific factors such as type of vegetation, age of the wetland, strength of influent wastewater, temperature and hydraulic variables (Frazer-Williams, 2010). The planted HSSF-CW column showed better BOD and TSS reduction compared to the unplanted. The results show that the activity of microorganisms in the root systems of the planted column was greater than in the unplanted plants. The reduced efficiency can therefore be due to the lack of the root system in the column, the relative reduction of microbial biofilms with active surface and a lack of oxygen via the root of plants.

Figure 4.32 - 4.34 respectively shows the mean observed and fitted NH_4-N, NO_3-N and PO_4^{3-} dynamics in the HSSF CW column. The input NH_4-N, NO_3-N and PO_4^{3-} concentrations were 63mg/l, 34mg/l and 11mg/l respectively. As can be seen from the figure, the nutrient reduction followed exponential trends. For NH_4-N, the values of rate constant, temperature coefficient and residual concentration that minimized the sum of error squared between the observed concentrations and the prediction of the modified first order kinetic model were 0.278/day

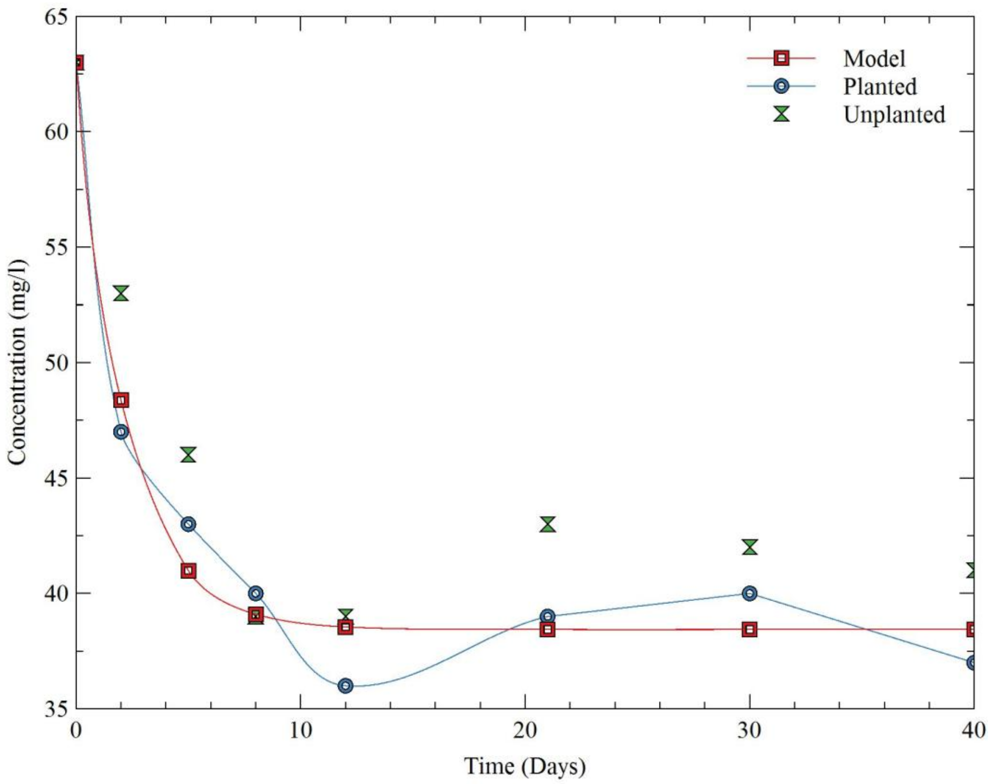


Figure 4.32 Measured and fitted NH_4-N dynamics during batch incubations in the HSSF-CW column.

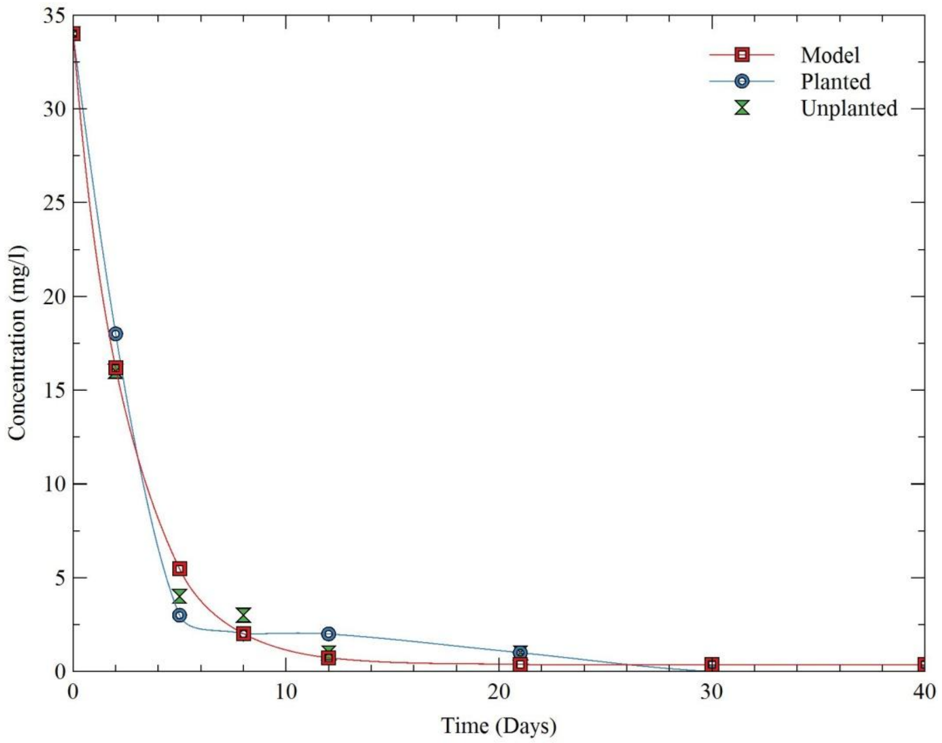


Figure 4.33 Measured and fitted NO_3-N dynamics during batch incubations in the HSSF-CW column.

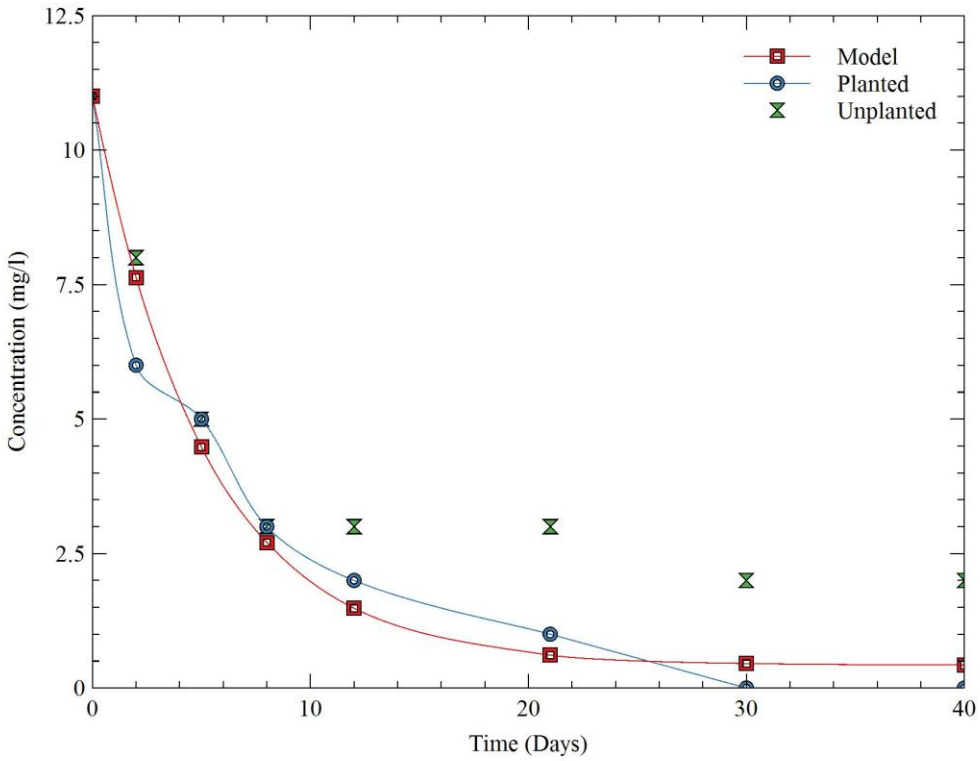


Figure 4.34 Measured and fitted PO_4^{3-} dynamics during batch incubations in the HSSF-CW column.

(15.01m/yr), 1.050 and 38mg/l respectively with a coefficient of determination for regression R^2 = 0.792. For NO_3-N, the values of rate constant, temperature coefficient and residual concentration that minimized the sum of error squared between the observed concentrations and the prediction of the modified first order kinetic model were 0.323/day (17.44m/yr), 1.015 and 0.36 mg/l respectively with a coefficient of determination for regression R^2 = 0.957. For PO_4^{3-}, the values of rate constant, temperature coefficient and residual concentration that minimized the sum of error squared between the observed concentrations and the prediction of the modified first order kinetic model were 0.306/day (16.53m/yr), 0.953 and 0.42mg/l respectively with a coefficient of determination for regression R^2 = 0.919.

Also, there are no removal rates reported in literature for nutrient in CWs using PKS as substrate. However, the rate constant and temperature coefficient obtained in this study do not differ significantly from the values reported in the literature. According to Kadlec and Knight (1996), the preliminary model parameters developed from the North American Wetland System Database were 18m/yr and 35m/yr for NH_4-N and NO_3-N respectively for rate constant at 20 ^oC reference temperatures and a temperature coefficient of 1.05 for both NH_4-N and NO_3-N. Cui *et al*., (2016) in their study on nitrogen removal in a HSSF CW estimated using the first-order kinetic model, reported that the area rate constants for NO_3-N and NH_4-N at 20 °C were 27.01 ± 26.49m/year and 16.63 ± 10.58m/year respectively and temperature coefficients for NO_3-N and NH_4-N were estimated at 1.0042 and 0.9604, respectively. Dzakpasu *et al*., (2014) in their assessment of an integrated constructed wetland for decentralized wastewater treatment in a rural community in Ireland reported a rate constant at 20 °C (k_{20}) of 15.4m/year and 5.1m/year for NH_4-N and NO_3-N respectively. The mean effects of temperature (θ) on the N removal rate constants were estimated to be 1.064 for NH_4-N and 1.004 for NO_3-N.

This finding of this study was consistent with previous reports that N removal in CWs is significantly influenced by temperature (Kadlec and Reddy, 2001). The very high residual concentration for NH_4-N is not consistent with the zero N residual concentrations in the literature. Kadlec and Wallace (2009) stated that treatment wetland systems can be assumed to have a theoretical background of zero for NH_4-N. Therefore, the residual concentration obtained must also be viewed with scepticism. Rate coefficient for PO_4^{3-} in the literature is scarce. Most design guidelines gave values for total phosphorus such as 0.11 - 0.18/day by Trang *et al*., (2010), 0.14/day by Tanner *et al*., (1995). Stone *et al*., (2002) reported that K_{20} values for TP ranged from 1.04 to 1.79 m/year. The somewhat negative effect of the temperature on the PO_4^{3-} treatment was surprising because it is well documented that phosphorus processes are not influenced by temperature (Stone *et al*., 2002; Kadlec and Wallace, 2009).

To examine the validity of the estimated model constants, simulation were conducted with model parameter estimates obtained during calibration and the results compared to values in the literature. The model parameter values are shown in Table 4.12.

Table 4.12 Rate constants, Temperature coefficient and residual concentration for simulations

Source		**BOD**	**TSS**	**NH_4-N**	**NO_3-N**	**PO_4^{3-}**
This Study	K_{20}	0.604	0.623	0.278	0.323	0.306
	Θ	0.995	1.093	1.050	1.015	0.953
	C*	43.23	33.77	38	0.36	0.42
Reed *et al.*, (1995)	K_{20}	1.104	-	0.2187	1.00	-
	Θ	1.06	-	1.04	1.15	-
	C*	6	-	0.2	0.2	-
Kadlec and Knight (1996)	K_{20}	2.166	0.801	0.648	0.926	0.168
	Θ	1.057	1.0	1.05	1.05	1.097
	C*	3.5+0.053Ci	7.8+0.063Ci	0	0	0.02

Figure 4.35 and 4.36 shows the measured and modelled BOD and TSS concentrations dynamics during the second batch incubation in the HSSF CW column. Lines with markers are the *k*–*C**model predictions using the parameter values obtained during calibration and the universal values proposed by Reed *et al.*, (1995) and Kadlec and Knight (1996). For BOD, the R^2 statistics indicated that the model based on the constants obtained during calibration explained 94.6% of the total variance in the observed effluent concentrations. For the values of Reed *et al.*, (1995) and Kadlec and Knight (1996), 84.9% and 84.7% of the observed concentration variability respectively were explained by the model. Also the model parameters obtained during calibration had the lowest SSQE of 7515, compared to the values of 65980 and 42217 obtained for Reed *et al.*, (1995) and Kadlec and Knight (1996) respectively. The very high SSQE for Reed *et al.*, (1995) and Kadlec and Knight (1996) showed significant variation between the measured and the modelled BOD concentration.

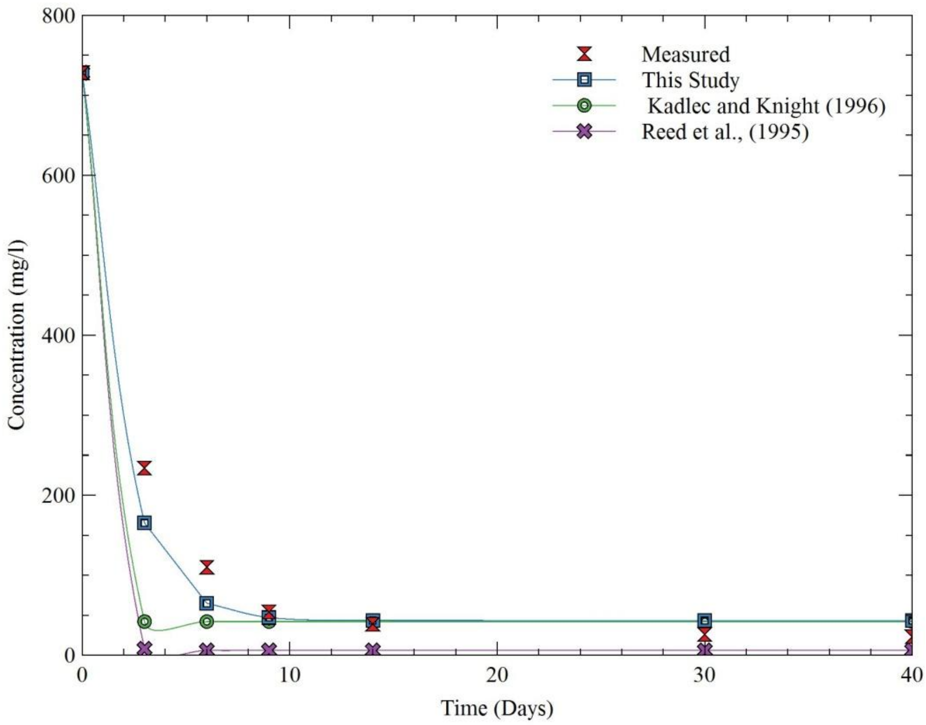


Figure 4.35 Measured and modeled BOD dynamics during second batch incubation in the HSSF-CW column.

For TSS, the R^2 statistics showed a very close agreement between the model constants, with 76.9% and 81.1% of the observed concentration variability explained by the model using the constants obtained during calibration and the values proposed by Kadlec and Knight (1996) respectively. The SSQE value of 1283 obtained for the parameters of Kadlec and Knight (1996) was also lower than the value of 4637 obtained for parameters of this study, which confirmed that the parameters of Kadlec and Knight (1996) were more accurate for predicting the TSS concentration.

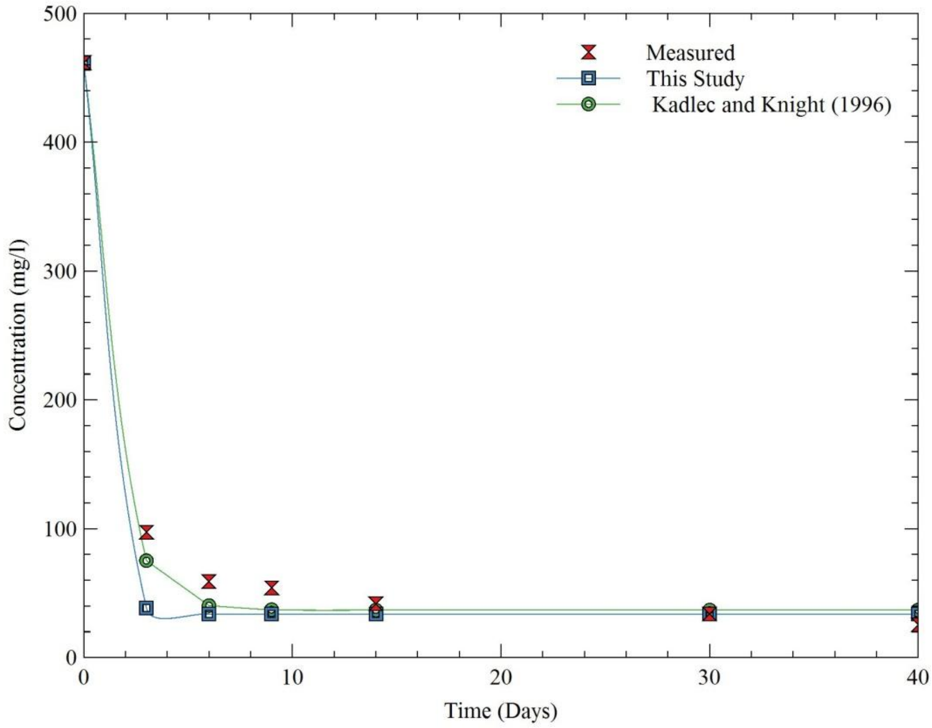


Figure 4.36 Measured and modeled TSS dynamics during second batch incubation in the HSSF-CW column.

Figure 4.37 shows the NH_4-N concentrations dynamics and the model predictions using the parameter values obtained during calibration as well as the universal values proposed by Reed *et al.*, (1995) and Kadlec and Knight (1996). The R^2 statistics indicated that the model based on the constants obtained during calibration explained 93.4% of the total variance in the observed effluent concentrations. For the values of Reed *et al.*, (1995) and Kadlec and Knight (1996), 97.6% and 85.7% of the observed concentration variability respectively were explained by the model. The SSQE value of 1142 was the lowest compared to the vales of 2559 and 5339 obtained for Reed *et al.*, (1995) and Kadlec and Knight (1996) respectively. This showed that the model parameters of this study was more accurate for NH_4^+-N prediction. The variation in the concentration data obtained compared to values obtained using the literature values was attributed to the significantly high value of background concentration obtained during the parameter estimation, compared to a theoretical value of zero in wetlands. The macrophytes in the columns were yound and so the background concentration will decrease with time.

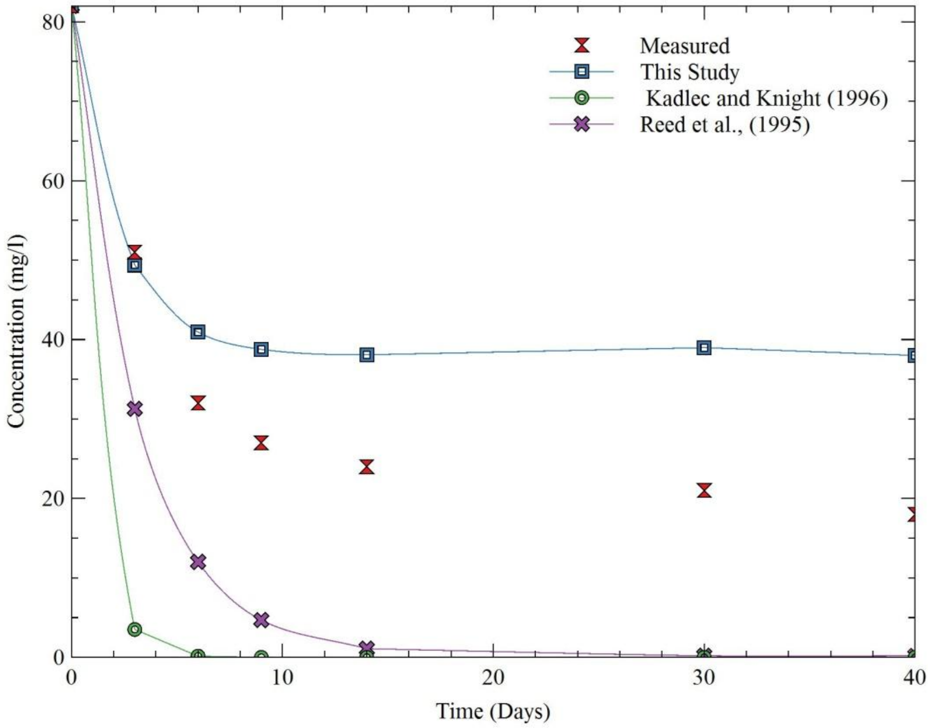


Figure 4.37 Measured and Modeled NH_4^+-N dynamics during second batch incubation in the HSSF-CW column.

Figure 4.38 shows the NO_3-N concentrations dynamics and the model predictions using the parameter values obtained during calibration as well as the universal values proposed by Reed *et al.*, (1995) and Kadlec and Knight (1996). The R^2 statistics indicated that the model based on the constants obtained during calibration explained 99.5% of the total variance in the observed effluent concentrations. For the values of Reed *et al.*, (1995) and Kadlec and Knight (1996), 88.3% and 89.0% of the observed concentration variability respectively were explained by the model. The SSQE value of 18 for the parameters of this study was also the lowest, compared to the values of 439 and 428 obtained for Reed *et al.,* (1995) and Kadlec and Knight (1996) respectively. It also showed that the model parameters of this study was more accurate.

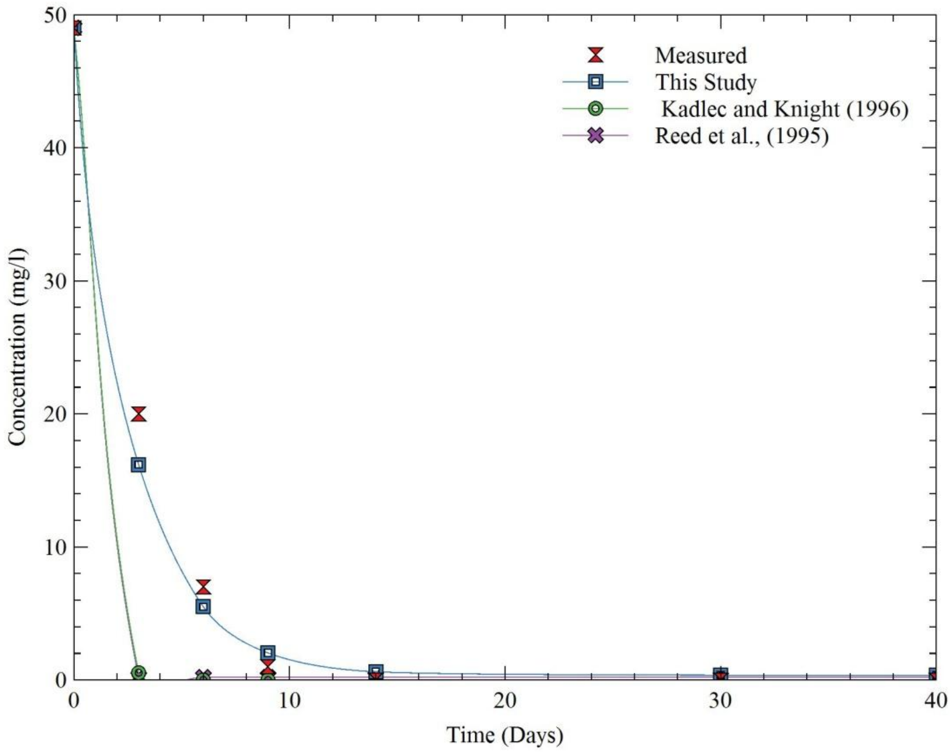


Figure 4.38 Measured and Modeled NO_3-N dynamics during second batch incubation in the HSSF-CW column.

Figure 4.40 shows the PO_4^{3-} concentrations dynamics and the model predictions using the parameter values obtained during calibration as well as the universal values proposed by Reed *et al.,* (1995) and Kadlec and Knight (1996). The R^2 statistics indicated that the model based on the constants obtained in during calibration explained 98.4% of the total variance in observed effluent concentrations. For the values of Kadlec and Knight (1996), 75.0% of the observed concentration variability was explained by the model. The SSQE value of 5 obtained for the parameters of this study was also lower than the value of 7 obtained for the parameters of Kadlec and Knight (1996), indicating more accuracy.

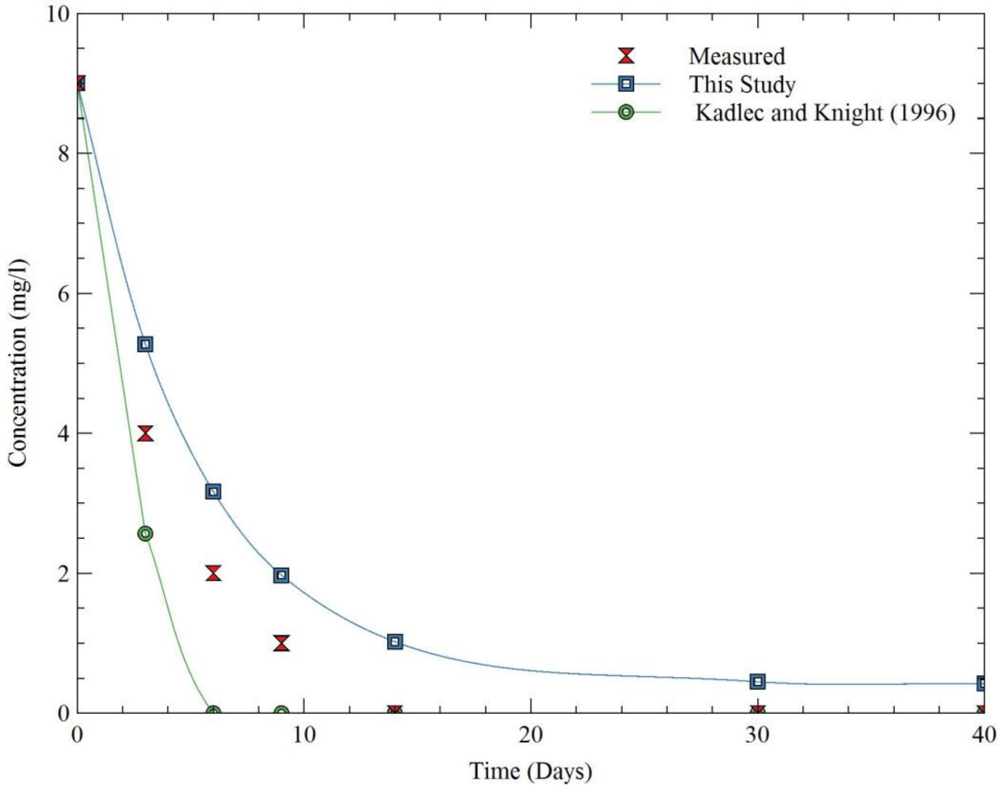


Figure 4.39 Measured and Modeled PO_4^{3-} dynamics during second batch incubation in the HSSF-CW column.

Model estimates described the overall constituent removal patterns well but deviated from observed values in some cases. The universal values proposed by Reed *et al.*, (1995) and Kadlec and Knight (1996) tended to overestimate contaminants removal in most cases. Evaluating the utility of the model constants by computing the R^2 goodness of fit, which measures the fraction of the total variability in the response that is accounted for by the model, is not enough. An $R^2 > 0$ value indicates a sufficient basis for accepting the model parameters. Unfortunately, a high R^2 value does not guarantee that the model fits the data well as is obvious in most of the figures above. The use of model constants for which the model does not fit the data well enough cannot provide good answers to the underlying engineering questions under investigation. SSQE was also used to compare validity outcomes, and it generally showed that the model parameters obtained during callibration were more accurate.

To examine the implications of the estimated design parameters on the sizing of a HSSF CW to treat slaughterhouse wastewater, the estimated parameters from the calibration and the universal values in the literature were used to predict the size of a field-scale system for Eke-Awka Etiti slaughterhouse as shown in Table 4.13. A mean temperature, depth and porosity of 28.9 ^{o}C, 0.5 m and 0.4 respectively were assumed and the input and output concentrations and flow parameter are as shown in the table.

Table 4.13 HSSF CW sizing using calibrated values, and values of Reed *et al.*, 1995) and Kadlec and Knight (1996).

	Q_{in} (m^3/d)	C_{in} (mg/l)	C_{out} (mg/l)	K_{20} (d^{-1})	θ	C* (mg/l)	Area (m2)	Diff (%)
BOD								
Calibration	2.0	622	50	0.604	0.995	23.0	53.65	-
Reed *et al.*, (1995)	2.0	622	50	1.104	1.06	6.0	14.23	-73
Kadlec and Knight (1996)	2.0	622	50	2.166	1.057	36.67	10.63	-80
TSS								
Calibration	2.0	457	65	0.623	1.093	25.6	17.40	-
Kadlec and Knight (1996)	2.0	457	65	0.801	1.0	36.60	33.64	48
NH_4-N								
Calibration	2.0	77	10	0.278	1.05	0	47.56	-
Reed *et al.*, (1995)	2.0	77	10	0.2187	1.04	0.2	65.82	28
Kadlec and Knight (1996)	2.0	77	10	0.648	1.05	0	20.41	-57
NO_3-N								
Calibration	2.0	42	20	0.323	1.015	0.36	20.38	-
Reed *et al.*, (1995)	2.0	42	20	1.0	1.15	0.2	2.01	-90
Kadlec and Knight (1996)	2.0	42	20	0.926	1.05	0	5.19	-75
PO_4^{3-}								
Calibration	2.0	13	5	0.306	0.953	0.42	137.7	-
Kadlec and Knight (1996)	2.0	13	5	0.168	1.097	0.02	68.04	-51

Using the parameters obtained for BOD during the model calibration in this study, the CW size was 53.65 m^2, 73% and 80% higher than the calculated areas using the values of Reed *et al.,* (1995) and Kadlec and Knight (1996) respectively. For TSS, the predicted area of the wetland based on the calibrated values of model constants was 48% lower than the predicted area using the value of Kadlec and Knight (1996). For NH_4-N, the area based on the constants of this study was lower than the values calculated using the values Reed *et al.*, (1995) by 28%, but was higher than the area calculated using the values of Kadlec and Knight (1996) by 57%. A very high difference was obtained between the calculated areas for NO_3-N. The calculated area from this study was lower than that of Reed *et al.,* (1995) by 90% and that of Kadlec and Knight (1996) by 75%. The predicted area for PO_4^{3-} based on the calibrated constants was 51% higher than the predicted area based on the constants of Kadlec and Knight (1996).

The very significant variations in the predicted CW sizes exposed the great risk in extrapolation beyond the calibration conditions. Kadlec and Wallace (2009) stated that "extrapolation from a wetland of one type to another is clearly not a reasonable step because the microbial communities, as well as the character and magnitude of the biogeochemical cycles, may differ markedly". Kadlec (1999) stated that difficulties arise when the model extrapolates outside of the calibrated concentration ranges, or for comparing design configurations. The findings of this study has established the fact that “universal” values of rate constants, as offered in many literature sources, does not exist as parameter values obtained from various operating wetland systems vary widely. Therefore, it is prudent to determine the model parameters before using them for design calculations.

4.4 Performance Evaluation of the Experimental Field-Scale HSSF CW

4.4.1 Plant Growth

The growth and development of the *Thalia geniculata* and *Colocasia esculenta* plants in the experimental field-scale horizontal subsurface PKS wetland were monitored during the study period. Maximum shoot heights of 1.48m and 0.92m were recorded for *Thalia geniculata* and *Colocasia esculenta* respectively. Similarly, maximum shoot density of 115shoots/m^2 was recorded. Rooting biomass was not quantified but physical inspection after three months of operation showed that the rooting mat established well, and had penetrated down to the wetland bottom.

4.4.2 Water Budget

The influent wastewater volumes during the study period were very variable and depended on the number of animals slaughtered. Higher volumes were recorded usually during weekends. Estimated values of the daily influent volumes ranged from 0.14 to 0.34m^3. The effluent volumes were not determined.

4.4.3 Ambient Conditions

The water temperature of the wetland followed the prevailing air temperature. Available data from the NIMET Synoptic Station, Awka, indicated mean monthly air temperatures in the range of 26.4°C to 30.2°C. The mean pH values obtained at the inlet and outlet of the PKS bed were 6.61 ± 0.33 and 7.14 ± 0.19 respectively. This increase of pH value may be due to the formation of some basic components in the bioremediation process, as also reported by Dhulap and Patil, (2014). However, Dhote and Dixit (2009) reported decreasing trend in water pH by using various aquatic macrophytes. The mean value of EC was 1789 ± 443.93µS/cm for wastewater influent and 1299 ± 338.80µS/cm for the effluent indicating a mean reduction of 26.59%.

4.4.4 Removal of Pollutants

Pollutant concentrations of the wetland influent (septic tank effluent) were variable throughout the time of the study. However, the effluent concentrations of most parameters remained steadily lower than the influent regardless of the fluctuations in the septic tank effluent. Table 4.14 presents the results of pollutant concentrations with the regular statistical indexes of mean, standard deviation, minimum and maximum, and the number of samples. The percentage reduction calculated on the basis of concentration for each parameter is also indicated.

Table 4.14 General results of treatment performance of the HSSF CW system

Parameter	Stat	Influent	Effluent	Removal Efficiency (%)
BOD (mg/l)	Mean	403.42±142.51	73.95±20.39	81.07
TDS (mg/l)	Mean	1384.72±356.96	832.64±179.79	39.06
TSS (mg/l)	Mean	326.08±85.38	58.57±17.63	82.12
NH_4-N (mg/l)	Mean	66.82±7.53	35.51±3.59	46.03
NO_3-N (mg/l)	Mean	33.48±5.05	20.28±3.22	38.13
PO_4^{3-} (mg/l)	Mean	10.04±2.11	5.84±1.46	40.92

Figure 4.40 shows the measured influent and effluent BOD concentrations in the experimental HSSF CW. Kadlec and Wallace (2009) classified influent BOD concentration for different treatment steps as 3 to 30 mg/l for tertiary treatment, 30 to 100mg/l for secondary treatment, 100 to 200mg/l for primary treatment and > 200mg/l for super-loaded systems. Thus with a mean influent BOD value of 403.42 ± 142.51mg/l the slaughterhouse wastewater falls under the category of systems with a very high BOD loading. The generally high BOD values can be attributed to the presence of blood in the slaughterhouse wastewater, which is known to be a very significant contributor to the strength of slaughterhouse wastewater. The mean outflow

BOD concentration was 73.95 ± 20.39mg/l, which indicated that the treatment performance did not meet the FEPA (1991) set limit of 50mg/l for wastewater discharge into the environment.

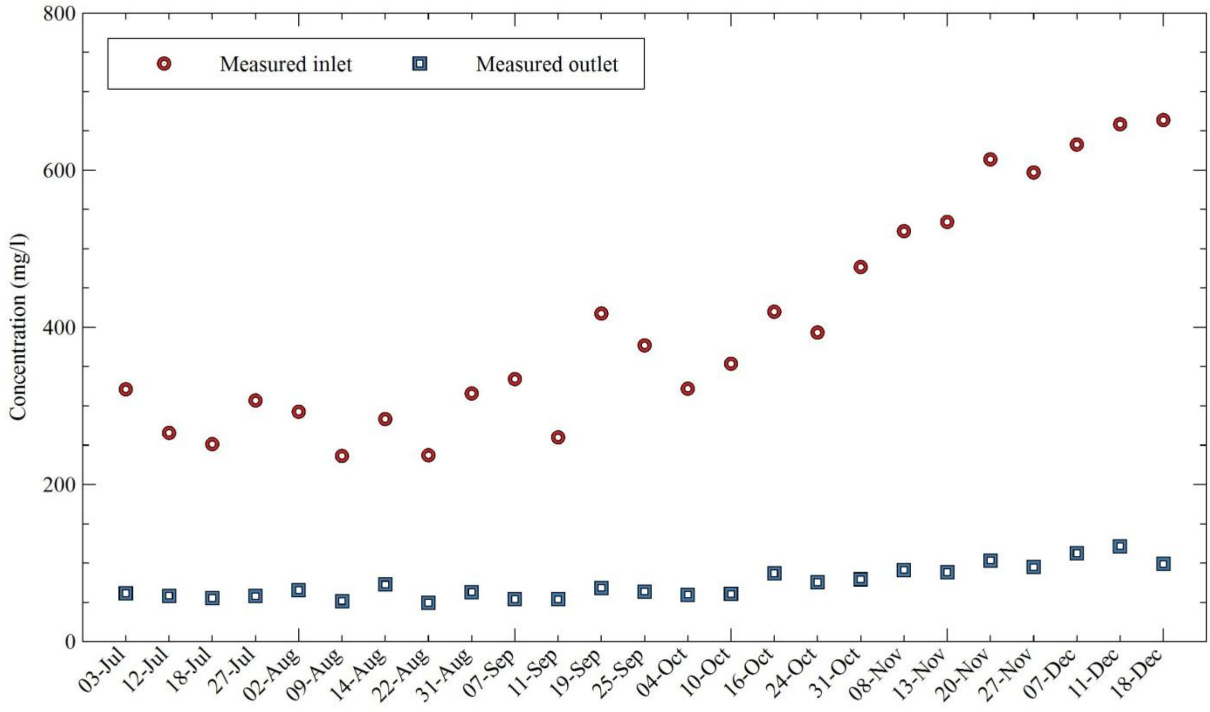


Figure 4.40 BOD concentration measured at the inlet and outlet of the HSSF CW

One-way ANOVA revealed a statistically significant difference (*p = 0.000*) between the mean influent and effluent wastewater BOD values. The 81.07% treatment efficiency of the PKS based experimental field-scale wetland was comparable to values reported in the literature for different types of constructed wetlands such as 82.5% (Adeniran *et al.*, 2014); 71 ± 6.2% (Mairi *et al.*, 2012); 82-85% (Badejo *et al.*, 2012); 74% (Kadlec and Knight, 1996); 79% (Haberl *et al*., 1995); 72% (Green *et al*., 1999) and 88% (Vymazal, 1999)

Figure 4.41 shows the measured influent and effluent TDS concentrations in the experimental HSSF CW. The mean value of TDS of the influent was 1384.72 ± 356.96mg/l with minimum and maximum values of 963mg/l and 1993mg/l respectively. The mean effluent TDS was 832.64 ± 179.79mg/l. The treatment performance achieved by the system is not of great importance as the influent wastewater concentrations were consistently below the 2000mg/l limit set by FEPA (1991) for effluent discharge. However, there was a statistically significant difference (*p = 0.000*) between the influent and effluent wastewater TDS mean values as determined by one-way ANOVA. The removal percentage of 39.06% was comparable to the value of 56.18% reported by Dhulap and Patil, (2014) for sewage treatment using subsurface flow constructed wetland.

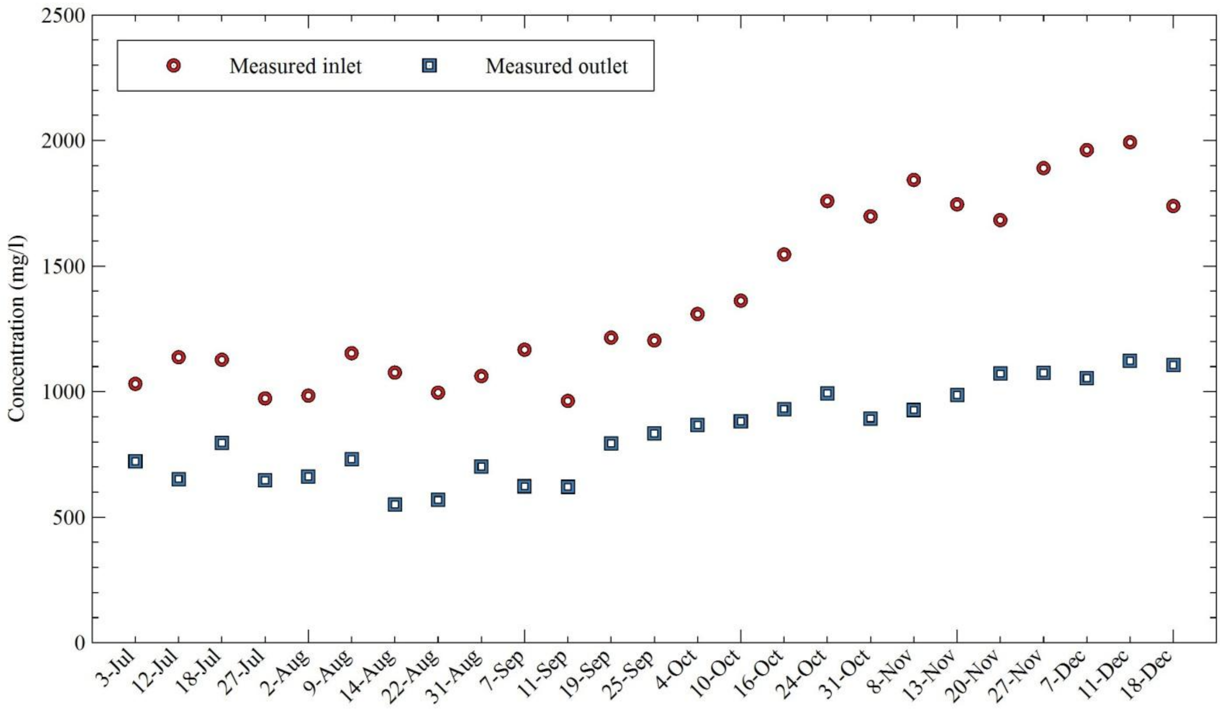


Figure 4.41 TDS concentration measured at the inlet and outlet of the HSSF CW

Figure 4.42 shows the measured influent and effluent TSS concentrations in the experimental HSSF CW. The mean value of TSS at the influent of the wetland was 326.08 ± 85.38mg/l, while the effluent mean was 58.57 ± 17.63mg/l. The mean TSS obtained in the study did not satisfy the 30mg/l limit set by FEPA (1991), but was significantly different (*p = 0.000*) from the influent. However, the 82.12% mean TSS removal percentage achieved by the PKS based wetland was consistent with ranges reported in the literature such as 86% by Molle *et al*., (2004) for purification performance of 54 reed beds in France. Nzabuheraheza *et al.*, (2012) reported removal percentage of 80.01%. Haberl *et al*., (1995) reported a removal percentage of 74% in Denmark, while Green *et al*., (1999) stated that for CWs operational in the UK, a removal percentage of 80% was common.

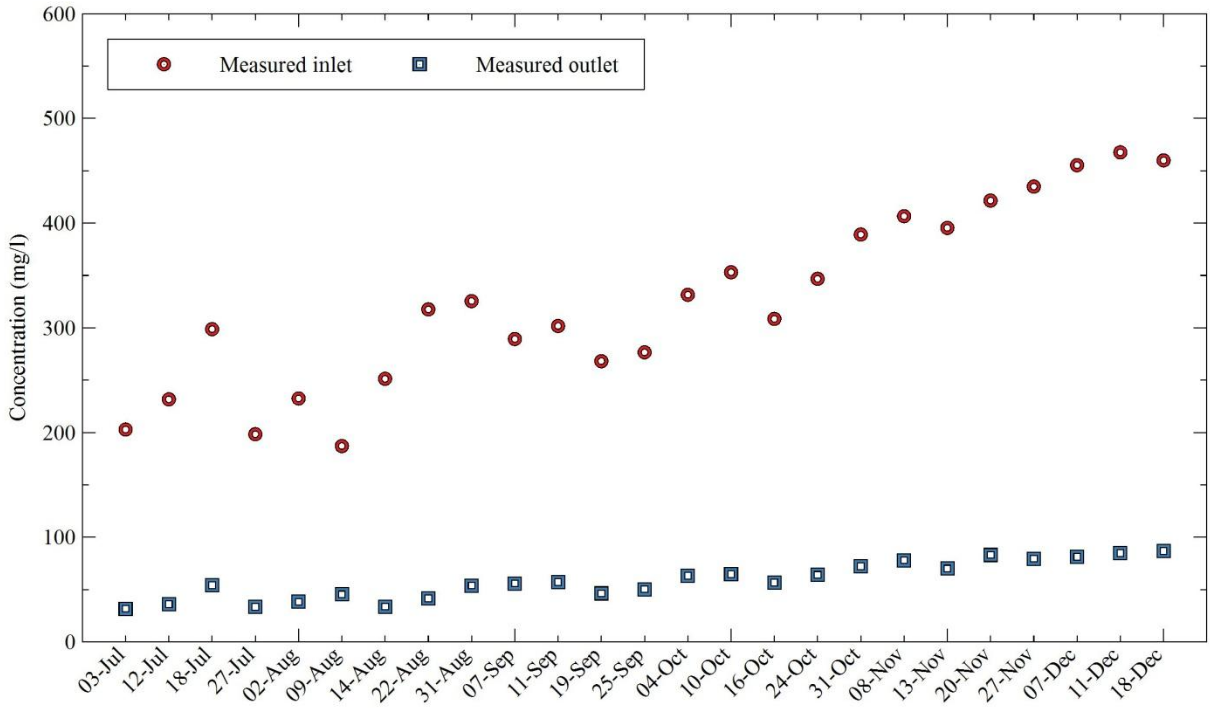


Figure 4.42 TSS concentration measured at the inlet and outlet of the HSSF CW

Figure 4.43 shows the measured influent and effluent NH_4-N concentrations in the experimental HSSF CW. The mean influent concentration of NH_4-N was 66.82 ± 7.53mg/l with minimum and maximum values of 52.17mg/l and 82.07mg/l respectively. The mean effluent concentration of NH4-N was 35.51 ± 3.59mg/l, which was much higher than the 10mg/l limit set by FEPA (1991) for effluent discharge. There was a statistically significant difference ($p = 0.000$) between the influent and effluent mean concentrations. There is a paucity of data on the treatment performance of full-scale PKS based horizontal subsurface flow wetlands, but the mean removal efficiency of 46.03 % obtained in the present study was comparable to the 54% were reported by Kadlec and Knight (1996) in their evaluation of the average performance of 70 North American wetlands treating domestic or agricultural effluent. It was also similar to the

30% removal reported for CWs in Europe by Haberl *et al*., (1995). Lower removal rates have been reported with age such as 14 % by Ran *et al*. (2004) and 11.2 % - 25.2 % by Kaseva (2003).

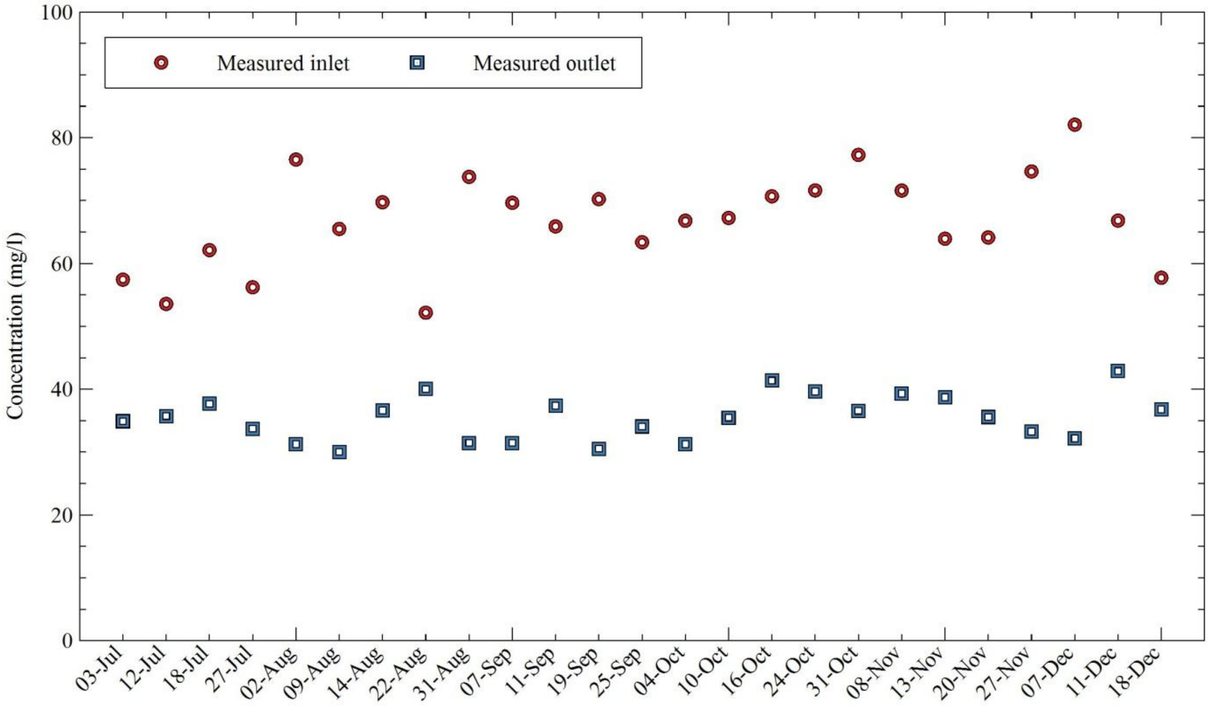


Figure 4.43 NH_4-N concentration measured at the inlet and outlet of the HSSF CW

As stated earlier, the main mechanism of NH_4-N removal from wastewater is the microbial mediated sequential nitrification - denitrification: the aerobic oxidation of ammonium to nitrite by ammonium oxidizing bacteria and the subsequent oxidation of the produced nitrite to nitrate by nitrite oxidizing bacteria (USEPA, 1999, Vymazal, 2007). The average removal rate in the present study can be attributed to the fact that the system was young and there was a sufficient oxygen supply to cover the need for microbial degradation of organic matter and nitrification of ammonium, and also to the fact that the plants were still maturing during the period leading to higher nitrogen intake, with the system likely to mature after 3-5 years (Kadlec *et al*., 2000).

Figure 4.44 shows the mean influent and effluent NO_3-N and PO_4^{3-} concentrations in the experimental HSSF CW. The mean influent concentration of NO_3-N was 33.48 ± 5.05mg/l with minimum and maximum values of 25mg/l and 43mg/l respectively. The mean effluent concentration of NO_3-N was 20.28 ± 3.22mg/l, which satisfied the 20mg/l limit set by FEPA (1991) for effluent discharge. There was a statistically significant difference (*p = 0.000*) between the influent and effluent mean concentrations. The mean influent concentration of PO_4^{3-} was 10.04 ± 2.11mg/l with minimum and maximum values of 7mg/l and 14mg/l respectively. The mean effluent concentration of PO_4^{3-} was 5.84 ± 1.46mg/l, which was slightly higher than the 5mg/l limit set by FEPA (1991) for effluent discharge. There was a statistically significant difference (*p = 0.000*) between the influent and effluent mean concentrations.

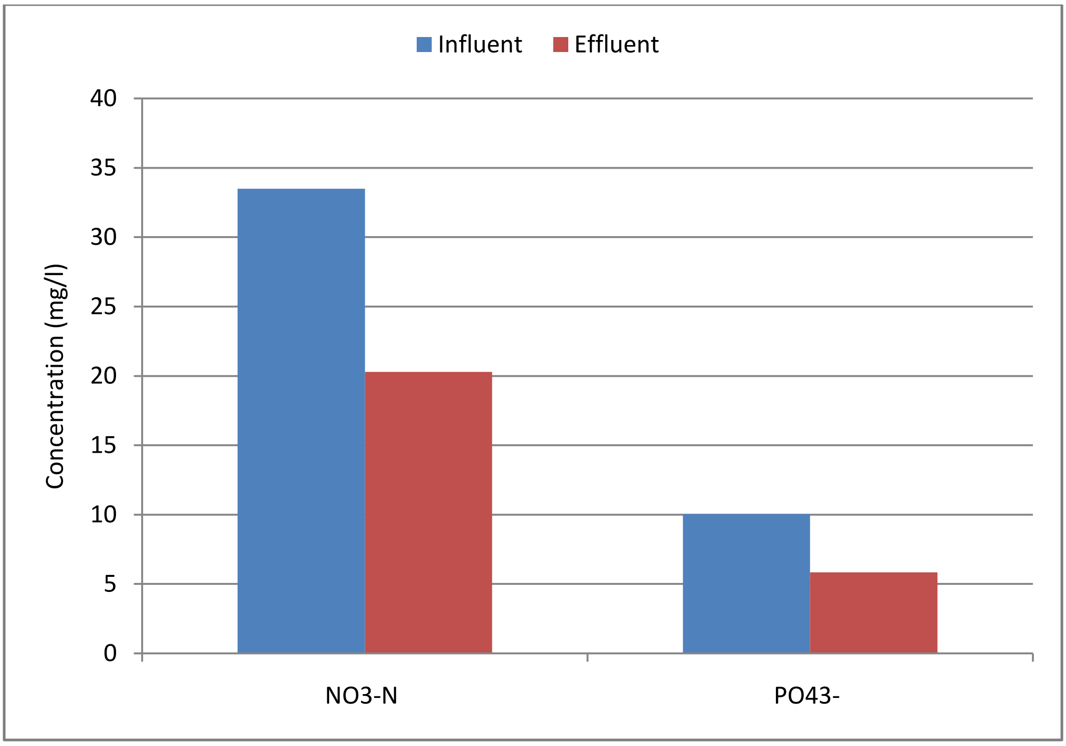


Figure 4.44 Mean NO_3-N and PO_4^{3-}concentration measured at the HSSF CW

No data exists on NO_3-N removal in a field-scale PKS-based horizontal subsurface flow wetland. However, the 38.13 % mean concentration reduction achieved by the wetland was not consistent with the ranges reported in the literature for other types of wetlands, such as the 61 % by Badejo *et al.*, (2012) and Kadlec and Knight (1996), 74.62 % by Dhulap and Patil (2014) and 56.37 % by Kassa and Mengistou (2014). The lower values in this study can be attributed to higher dissolved oxygen concentration, leading to aerobic oxidation of ammonium to nitrite by ammonium oxidizing bacteria and the subsequent oxidation of the produced nitrite to nitrate by nitrite oxidizing bacteria.

Also no data exists on PO_4^{3-} removal in a field-scale PKS based horizontal subsurface flow wetland. However, the mean PO_4^{3-} removal efficiency of 40.92 % recorded in the study was lower than the values reported for other types of wetlands, such as 51.1 % to 52.0 % by Chang *et al.,* (2012), 57.81 % by Dhulap and Patil (2014), and 81.0 % Badejo *et al.*, (2012). It is a widely acknowledged fact that adsorption to soil or substrata is suggested to be the principal mechanism that contributes too much of P removal (Wood, 1990, Vymazal, 2007). The lower removal rate obtained in the present study suggested a poor adsorption capacity of the PKS substrate. However, this is not conclusive as decreased phosphate reduction can also be associated with the release of stored phosphorus by the decaying litter.

The experimental PKS based field-scale HSSF CW in this study effectively removed BOD and TSS from the pre-treated slaughterhouse wastewater, but not to concentrations below that prescribed by the regulating authority in Nigeria. Notwithstanding, utilizing PKS in a field-scale constructed wetland significantly improved the effluent quality. The system was effective in removing NO_3-N and PO_4^{3-} from the pre-treated slaughterhouse wastewater to concentrations below the set limits in Nigeria. NH_4-N removal did not satisfy the limits set by the regulating authority, but was significantly reduced. Therefore, it is safe to say that utilizing PKS in a field-scale constructed wetland significantly improves the effluent quality in terms of nutrients.

4.5 Results of Tracer Experiment

The tracer study was performed to gain an understanding of the hydraulics of the field-scale experimental HSSF CW with PKS as substrate. Section 4.5.1 presents the data and graphs of the tracer test, while section 4.5.2 presents the result of mathematical models applied to provide parameters of the system.

4.5.1 Tracer Dynamics in the HSSF CW

a) Water Budget

The water balance of the HSSF CW was dominated by the inflow and outflow of borehole water as there was no rainfall during the period. The average inflow delivered over the 13 days of experiment to the system was 750L/d and the average outflow measured was 669.08L/d. The difference was 80.92L/d which was lost by ET, representing 10.8% of the total inflow, or 4.7mm/day. This value was higher than the average value of 3.9mm/day reported for South Florida by Abtew and Obeysekera (1995), and the daily average evapotranspiration rate of 3.5mm/day in November 2009 and 3.3mm/day in January 2010 reported in South Africa by Mangistu *et al*., (2014). There is a paucity of data on evapotranspiration rates from constructed wetlands in the tropical environment of Nigeria; however the high evapotranspiration rate was expected due to the prevailing intense solar radiation during the study period.

b) Internal Tracer Flow in the PKS Bed

Figures 4.45 - 4.47 presents the results of measurements through classical Tracer Breakthrough Curves (TBC) of the NaCl concentration at the 9 sampling ports within the PKS bed. Figure 4.45 presents the TBC for the transect at 1m from the inlet distribution pipe, while Figures 4.46 and 4.47 presents the TBCs for the transects at 3m and 6m respectively.

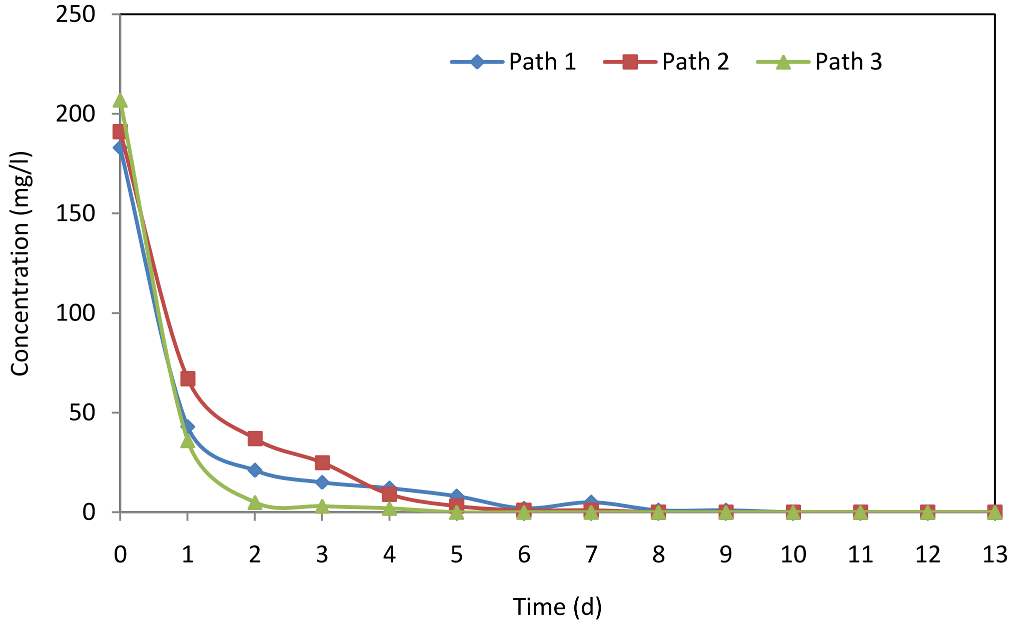


Figure 4.45 NaCl tracer response curves for 3 sampling points at the first transect

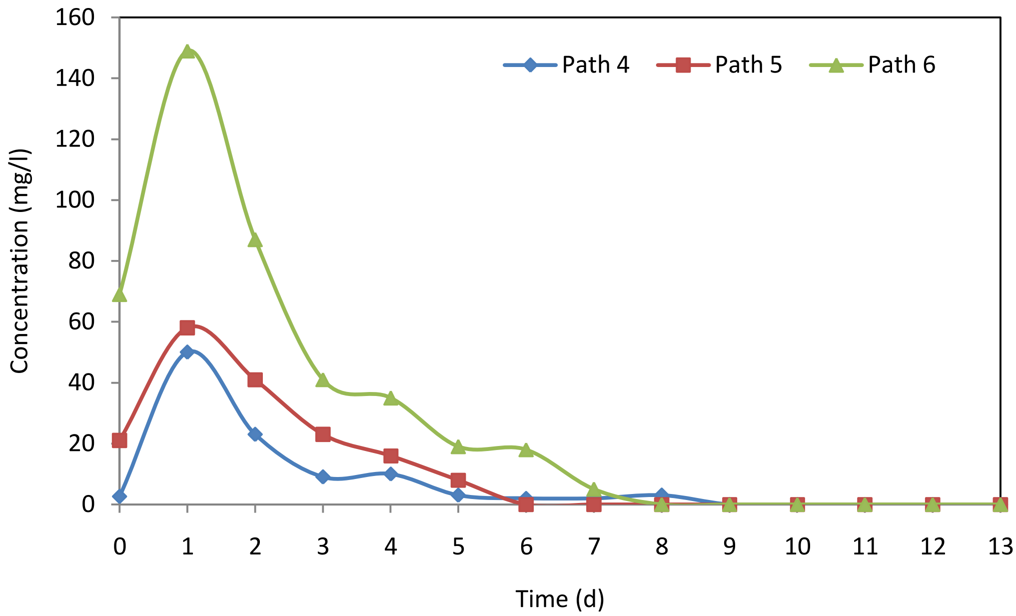


Figure 4.46 NaCl tracer response curves for 3 sampling points at the second transect

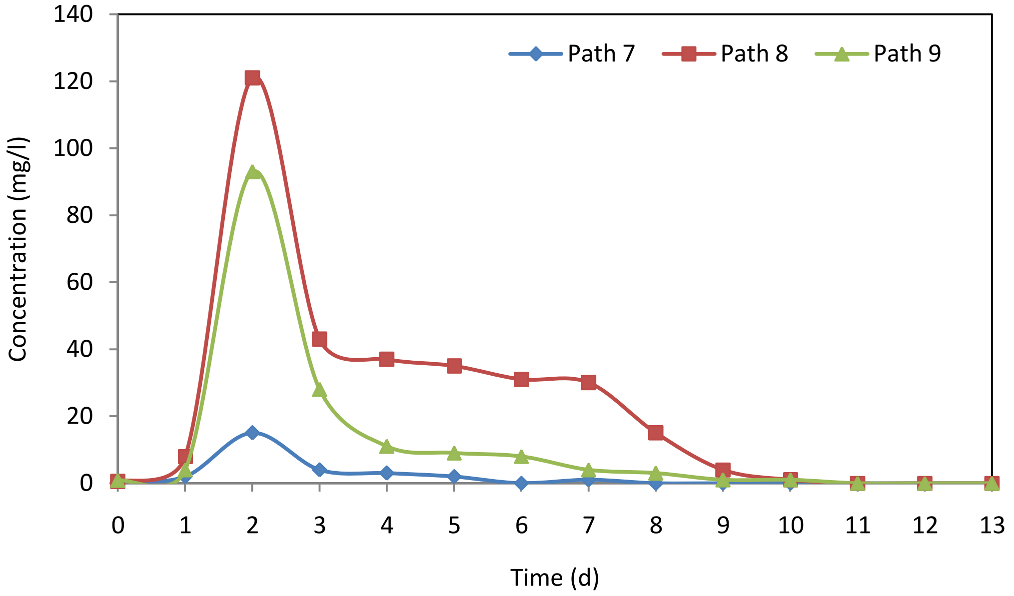


Figure 4.47 NaCl tracer response curves for 3 sampling points at the third transect

From the graphs it can be seen that all obtained curves had a general regular bell shape, corresponding to theoretical expectations. There was a visible decrease in tracer concentration over length likely due to mixing or dispersion conditions. By the second day, the tracer gradually faded away at the inlet side and continued to rise at the outlet side. For the first transect (1m from inlet) the highest tracer concentration was observed at path 3. It is thus clear that more tracers arrived at path 3, exposing the existence of preferential flow paths because same tracer peaks were not reached at the same time as a homogeneous system should do. The figures clearly show a preferential flow path to the right of the PKS bed (along paths 3, 6 and 9). This can be attributed to the higher vegetation density along paths 1, 4 and 7 as can be seen in Figure 4.48, which invariably translated to have a higher below ground biomass.

Figure 4.48 Full-scale HSSF CW showing higher macrophyte density on the left path

Weaver *et al.*, (2003) in their study of water flow patterns in subsurface flow constructed wetlands designed for on-site domestic wastewater treatment stated that the presence of plants caused preferential water flow around root masses. Also Bodin (2013) in their study of the effects of vegetation, hydraulics and data analysis methods stated that vegetation patterns controlled most of the water flow paths as the heterogenic distribution and density of vegetation stands lead to short-circuiting paths and dead zones and thus they argued that construction of wetlands should prioritize vegetation establishment more than the design of bottom topography.

c) Exit tracer response curve

The TBC obtained from the outlet was also a time-delayed bell-shaped curve as expected. The flow behaviour was neither ideal nor perfect, because the peak did not appear at a time equal to the nominal residence time ($\tau_p = \tau_n$), which was not surprising. The long tail is indicative of the existence of dead zones. It can also result from tracer adsorption-desorption. Irregularities of a second peak on day 7 can easily be recognized on the graph in figure 4.49. This can be attributed to a pocket of water that moved at a different rate than the main stream, which underscores the heterogeneity of the studied system.

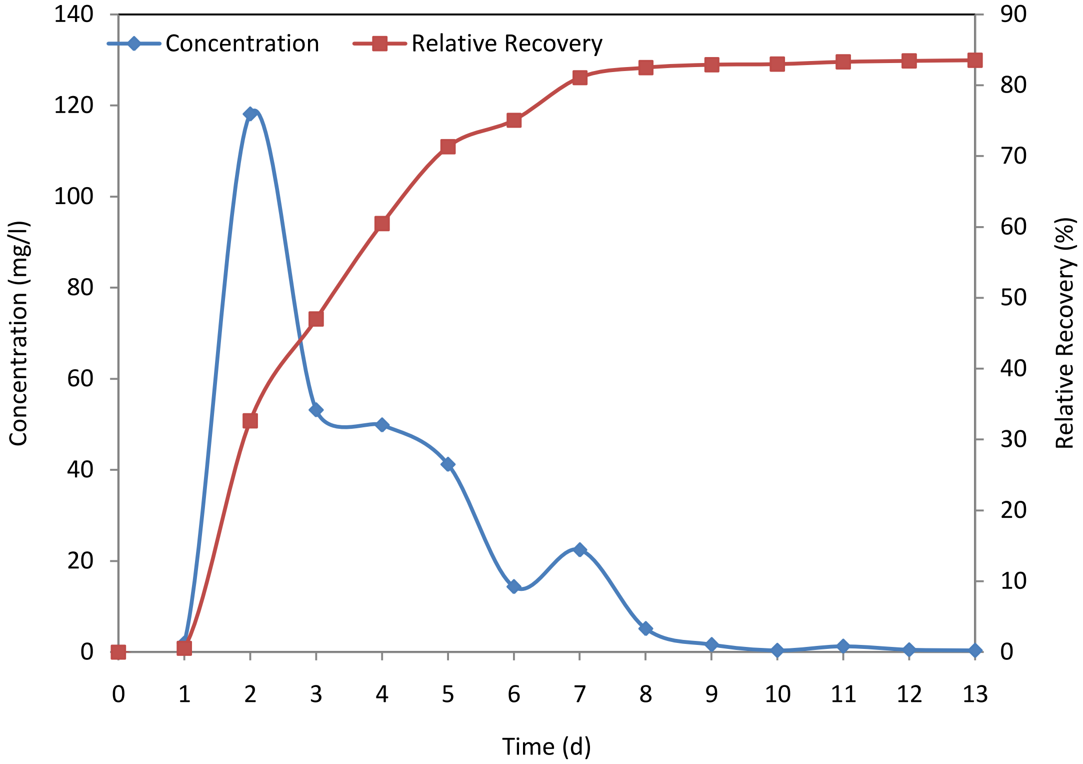


Figure 4.49 Tracer response curve at the exit of the wetland and relative recovery of tracer

The NaCl mass balance was checked, by comparing the added mass to the mass found in the exit flow. From the recovery curve in Figure 4.51, 83.5% of the tracer mass was recovered at the outlet until the monitoring was stopped on the 25th January after thirteen days of measurement. Salt sorption/desorption to plants or substrate bed and other losses in the system such as biological uptake are some of the reasons for low tracer mass recovery. According to Kadlec and Wallace (2009), mass recoveries of 80-120% are indicators of successful wetland hydraulic tracer studies. Thus, the amount of mass recovered in this study can be classified as high, indicating that the present method with sodium chloride was adequate for describing hydraulics in the studied wetland.

d) Residence time distribution (RTD) characteristics

In order to quantify wetland hydraulics, the RTD data from the hydraulic tracer experiment was analyzed using the method of moments, and other regular parameters were calculated as described in the theoretical background. The RTD characteristic is presented in Table 4.15.

Table 4.15 Data on the residence time distribution of the HSSF CW

Parameters	Symbol	Formula	Unit	Value
Length	L		M	7.2
Width	W		M	2.4
Depth	D		M	0.5
Volume	V	L * W * D	m^3	8.64
Porosity	E	Measured		0.37
Nominal Volume	V_n	$V * \varepsilon$	m^3	3.2
Inflow	Q	Measured	$m^{3/}$day	0.75
Nominal Detention Time	τ_n	V_n/Q	Days	4.27
Tracer Peak Time	τ_p	Measured	Days	2
Mean Detention Time	τ		Days	3.63
Hydraulic Efficiency	λ	τ/τ_n	%	0.85
Variance	σ^2		$days^2$	3.39
Dimensionless Variance	σ^2_Θ	σ^2/ τ^2		0.26
NTIS	N	$1/ \sigma^2_\Theta$		3.9
Active Volume	V_a	$\tau * Q$	m^3	4.84
Dead Space		$V-V_a$	m^3	3.8
Peclet Number	Pe			6.7
Dispersion Number	P_D			0.15

It is clear from the table that the nominal detention time was about 21% larger than the tracer detention time, which suggested short-circuiting paths that allow certain elements of flow to pass through the treatment system ahead of, or faster than, the nominal detention time of the system, which is indicative of obstructions, stagnant regions, and velocity gradients.

Hydraulic Efficiency (λ), which describes how well the incoming water distributes within the bed, was on the high end of the typical range of 0.15 to 1.38 reported by Kadlec and Wallace (2009) for subsurface flow wetlands. The λ-value in the current study was similar to the value of 0.83 reported by Speer *et al*., (2009), and was higher than the value of 0.41 reported by Dierberg *et al*., (2005) for a gravel bed wetland. Garcia *et al*., (2004) stated that hydraulic efficiency can be categorised as “good hydraulic efficiency” when λ is > 0.75 (or 75%). Persson *et al*., (1999) gave a range of $\lambda \geq 0.5$ for a good hydraulic efficiency. Therefore, the λ-value from the present study is indicative of a wetland of effective hydraulic condition. Higher hydraulic efficiencies can correlate to higher treatment efficiencies as well, mainly due to the extended retention time. However, Wang and Jawitz (2006) criticized the use of λ-values derived from RTDs with multiple peaks, since determining accurate t_p values from such RTDs and few measured data points in the peak region is associated with uncertainty.

The dimensionless variance (σ^2_Θ) of the tracer response curve corresponds to the degree of mixing in the wetland. It is a measure of the spreading of the concentration pulse after travel through the wetland (Kadlec, 1994). This parameter is zero for a plug flow system, and unity for a totally mixed system. Thus the constructed wetland in this study is 26% of the way from plug flow to completely mixed. Dead space was calculated by considering the volume of a reactor that would produce a residence time equal to the mean residence time obtained from the analysis of the RTD curve. The resultant "reduced" volume is termed effective volume and the percentage of dead volume was obtained by subtracting this quantity from the actual reactor volume, without porosity reduction. Moreno (1990) explained dead spaces accounted for anywhere from 10 to 21% for facultative stabilization ponds studied. El Hamouri *et al*., (2007) reported a dead zone volume of 30% for three HSSF CWs.

The active volume (V_a) of about 56% from the present study lies in the middle of the range reported in other published hydraulic tracer studies in horizontal subsurface flow wetlands (Dierberg *et al*., 2005; Kadlec and Wallace, 2009; Speer *et al*., 2009; Keefe *et al*., 2010). According to Thackston et al. (1987), wetlands with $0.5 \leq V_a \leq 0.75$ have a moderate amount of dead zones, whereas those with values above 0.75 have a small amount. This means that the wetland in the present study, based on measured data, contained moderate amounts of dead zones, which can be attributed to the existence of preferential flow paths induced by the non-uniform vegetation density in the system. Dispersion is the term used to describe the evolution of a transition zone that develops as fluids move in the direction of a uniform composition. Dispersion number (P_D) values for horizontal subsurface flow wetlands are generally in the range 0.009 to 0.48 (Kadlec and Knight, 1996). Cothren et al., (2002) found the range of dispersion in a bench scale horizontal subsurface flow wetland to be from 0.107 to 0.345. Chazarenc *et al*., (2010) fitting experimental data with PFD model and reported System dispersion numbers in the range of 0.14 to 0.36. Thus, the dispersion number calculated from the RTD of the tracer in this study was on the low end of the range. This can be attributed to the fact that the system received low water velocities which limited the spreading and mixing of incoming water.

4.5.2 Mathematical model applications and data fitting

Figure 4.50 presents the graphical results of the calculation for the field-scale experiment. Blue line is the measured data of normalised tracer concentration versus time; green line is the normalised tracer concentration based on the mathematical solutions of the *TIS* equation adjusted using the least squares optimization method; red line is the normalised tracer concentration based on the solution of dispersed plug flow model adjusted using the least squares optimization method.

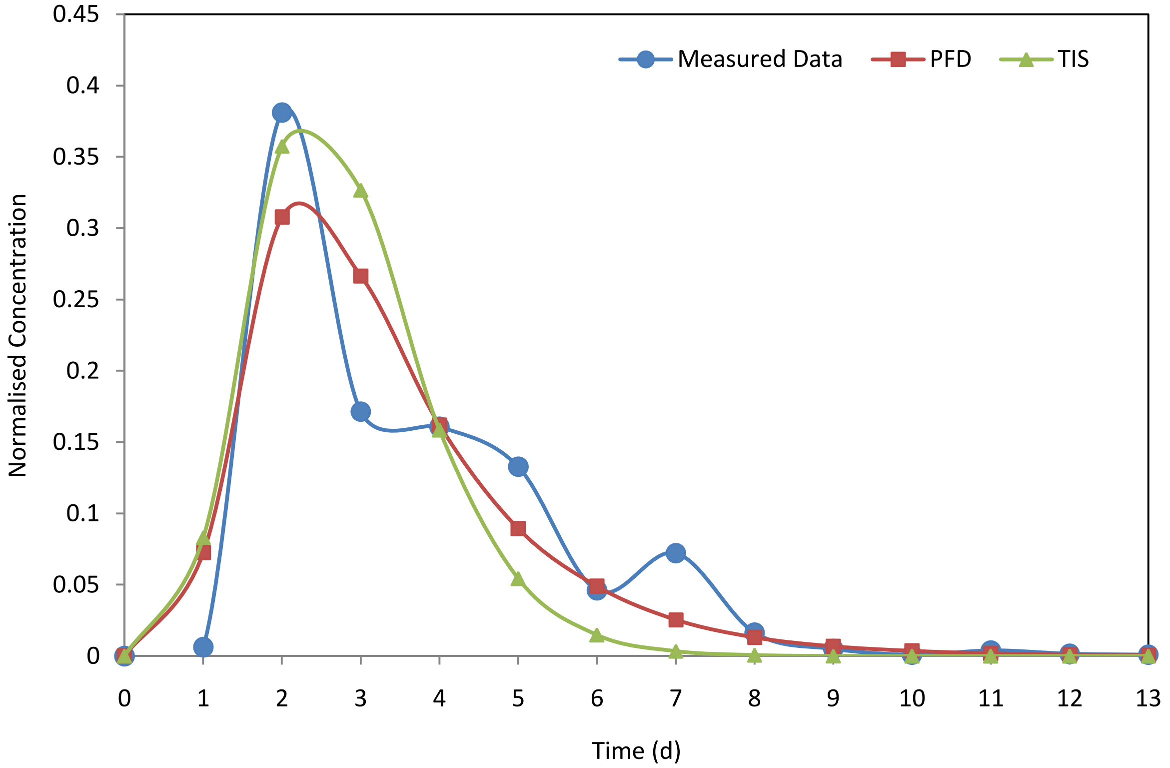


Figure 4.50 Field data, tanks in series and dispersion model solutions and adjustments from tracer experiment.

According to Wang and Jawitz, (2006), when measured data are well represented by a selected model, reliable modelled parameter values should be obtained. The least square error method applied to sum the square of the difference between the measured data and *TIS* model curve (green line) showed a reasonable fit over the entire response and provided new values of the system parameters indicative of the general hydraulic behaviour. The red line showed the solution of the *PFD* model equation and the model fits reasonably well (comparable to the TIS model). The tracer peak was similar between the two models. The lower sum of squared error (SSQE) value of 0.023 for the *PFD* model compared to the 0.043 obtained for the *TIS* model showed that the *PFD* model produced a more acceptable detention time distribution, although the difference was not statistically significant ($p = 0.258$). Furthermore, a plot of the model predicted concentrations versus measured concentrations (Figure 4.51 and 4.52) showed that the

PFD model adjusted with least squares optimization was able to describe 85.2% of the variability in the measured outlet tracer concentration, while the *TIS* model adjusted with least squares optimization was able to explain 78.4% of the variability in the data set. This showed that the *PFD* model described very well the system under investigation and thus provided new and different information on the hydraulic behaviour than the *TIS* model did.

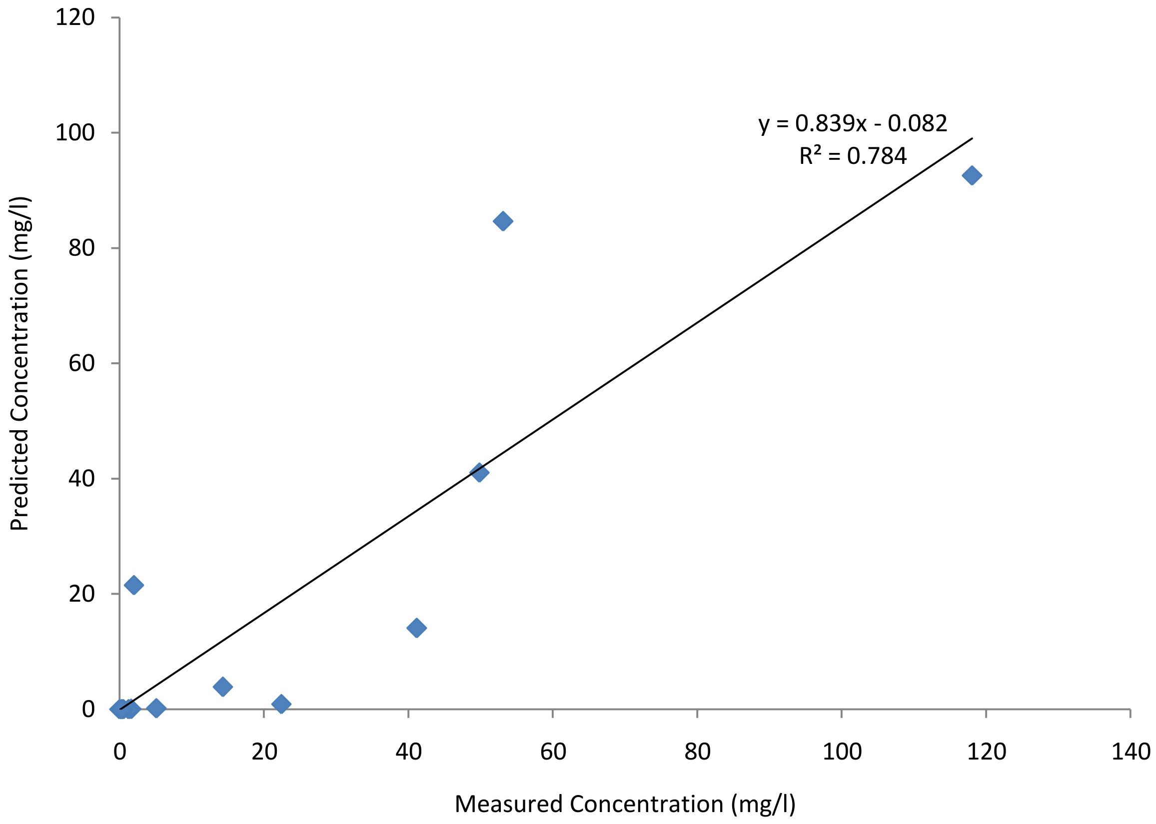


Figure 4.51 TIS predicted tracer concentration vs measured concentration

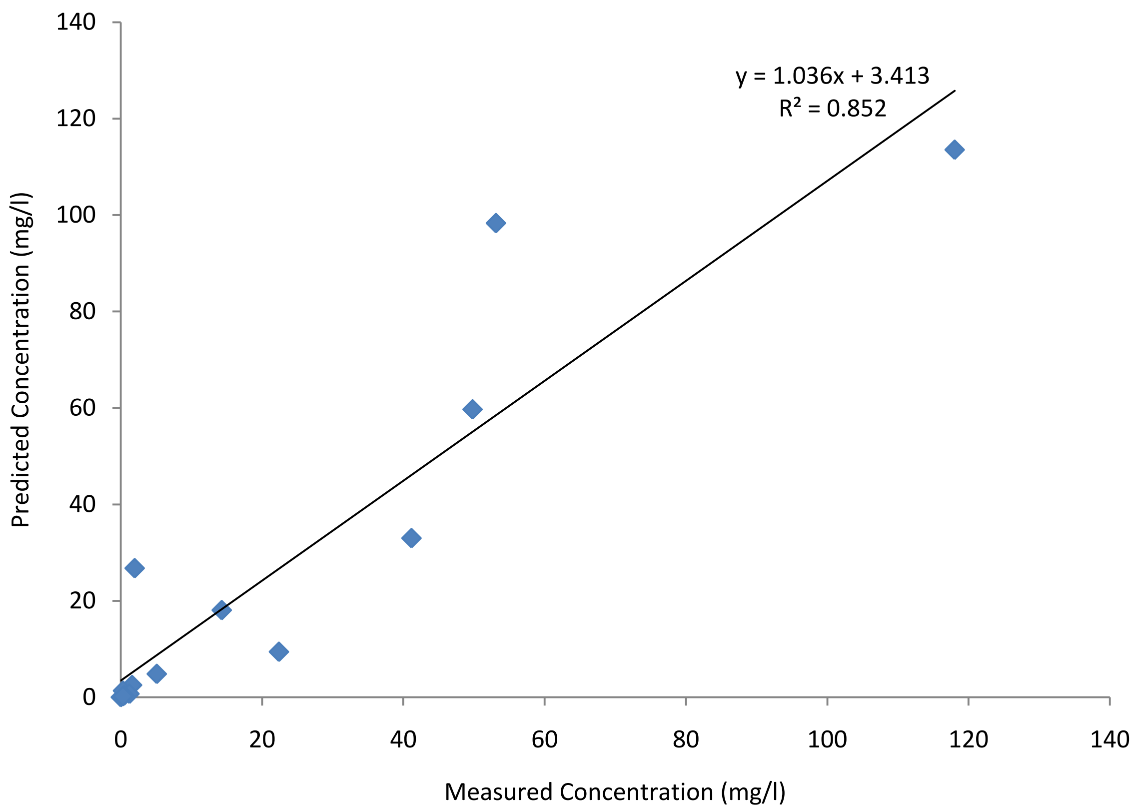


Figure 4.52 PFD predicted tracer concentration vs measured concentration

The system parameters values obtained by the solutions of mathematical models and their calibration from the tracer experiment are presented in Table 4.16.

Table 4.16 Parameter values obtained by models and calculation methods for the full-scale constructed wetland

Parameter	Symbol	Unit	MM	TIS	PFD
Tracer detention time	T	Days	3.64	2.81	3.28
NTIS	N		3.90	6.38	-
Dispersion Number	P_D		0.149	-	0.136
Peclet Number	Pe		6.7	-	7.35

The table firstly presents hydraulic parameters obtained from the method of moments (MM), secondly, the results of the *TIS* model calibrated with the least squares optimization method, and finally the results of the hydraulic parameters of the *PFD* model adjusted with least squares optimization.

The values of most parameters were similar. The new value of tracer detention time (τ) obtained from the calibration of the *PFD* model was systematically lower than the value calculated by the Moment Method (MM). For the *TIS* model, the tracer retention time was lower than the value calculated by the Moment Method. When the nominal retention time value of 4.27days is compared to the times obtained by the models, important differences are visible. These shorter retention times obtained by the model calculations can be attributed to the existence of bypass (or short circuit) streams. The number of completely mixed tanks in series was increased by the *TIS* model to 6.38 tanks. The dispersion number (P_D) is one of the parameters obtained from the *PFD* model. Levenspiel, (1972) explained that reactors with dispersion numbers greater than 0.2 exhibit "a large amount of dispersion;" whereas, a dispersion number of 0.025 represents flow somewhere between plug flow and mixed flow. A very small amount of dispersion (< 0.002) results in near plug flow. Thus a dispersion number of 0.136 shows that there is a moderate dispersion in the system.

The findings of this study has reinforced the obvious submissions that considering CW beds as a homogenous system, having uniform flow through the whole cross sectional area is a wrong assumption. The *PFD* and *TIS* models have been used to provide reasonable approximations of the varying degrees of hydraulic characteristic of non-ideal flow. The models provided more accurate detention time and number of tank in series which are important for design considerations and the general evaluation of the hydraulic pattern (being closer to plug flow or completely stirred). The *PFD* model provided additional information on dispersion within the PKS bed. It has demonstrated that flow rates are not homogeneous with cross sectional area of the bed, as the total flow is not delivered homogeneously, buttressing the fact that CWs cannot be classified as ideal flow systems. Therefore, design models that adequately characterize the complex processes that occur in CWs, while incorporating atmospheric interactions such as precipitation, evaporation, and transpiration, which can produce a secondary hydraulic regime that can influence retention times and invalidate steady state theoretical conditions, are needed.

4.6 Results of Computational Fluid Dynamics Modeling of the Field-Scale HSSF CW

The two-dimensional velocity fields and streamlines for the PKS based experimental field-scale HSSF CW that was obtained in the first stage of simulation is presented in Figure 4.53. Firstly the direction of flow from the inlet towards the outlet can be identified. Furthermore the parts of the wetland where the flow intensity is greater can be distinct. Longer arrows indicate that the flow intensity in these areas is higher than in neighbouring areas.

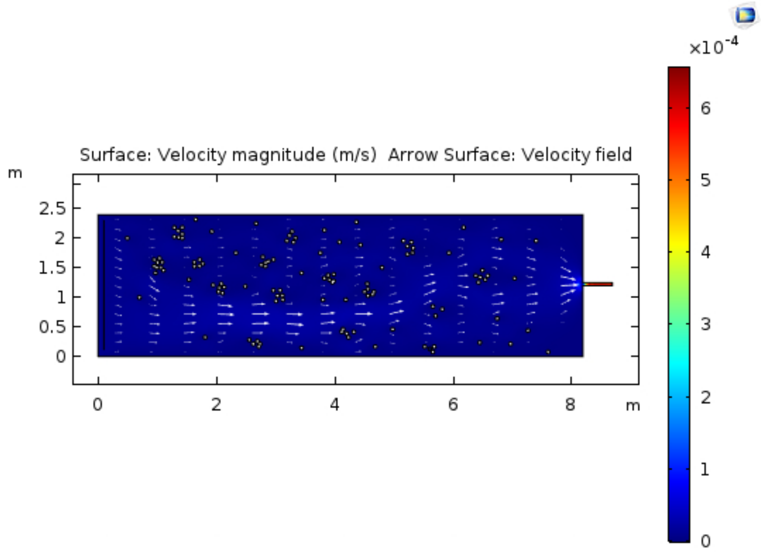


Figure 4.53 Velocity Profile of the Field-scale HSSF CW

From the 2D plot, it can be seen that dead zones (i.e., areas with very low flow velocity) occurred where the vegetation density was highest which would invariably decrease the hydraulic performance by inhibiting water exchange. A region of short-circuited flow (i.e., a region of high flow velocity) was observed on the right side of the wetland, which would lead to a reduced residence time and flow uniformity and thereby decreased the hydraulic performance and treatment efficiency. Hydraulically, treatment efficiency is considered satisfactory as long as the system is well mixed and the physical characteristics are uniform across the wetland perpendicular to the flow (Williams and Nelson, 2011). The preferential flows generated due to non uniform distribution of vegetation can be seen more clearly, and the occurrence of short-circuited flows generated in the system will likely result to poor treatment efficiencies, especially for parameters for which adsorption occurs.

To distinguish the tracer behaviour, the two interphases (Laminar flow and Transport of diluted species in porous media) where coupled and the profile of the concentration in 2 dimensional geometry at various times is presented in Figures 4.54

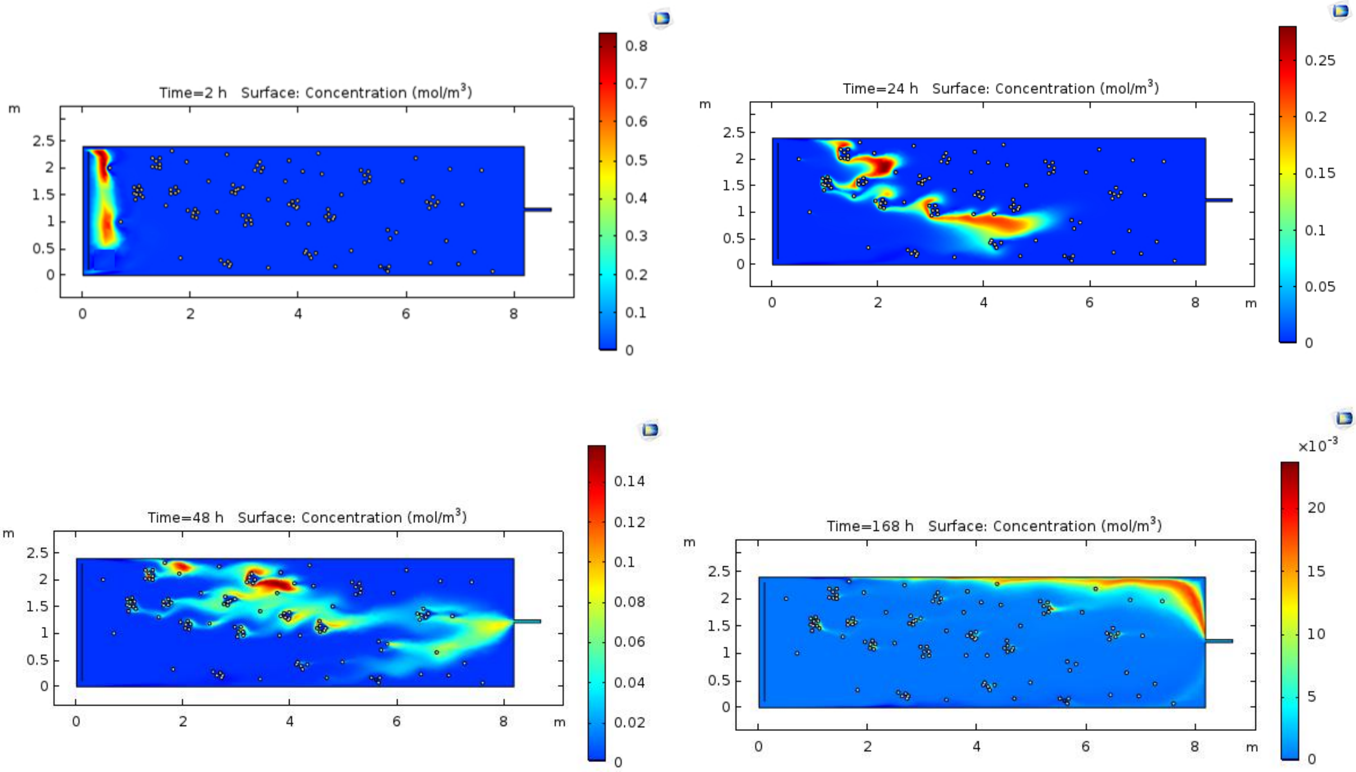


Figure 4.54 Spatial distribution of tracer concentration at different times after injection

The behavior of the tracer concentration provided adequate information about the wetland as well as for the tracer. The spreading of the tracer followed the preferential flow path already identified in the first stage of simulation. Furthermore, with the simulation of tracer transport, it was possible to verify the hydrodynamic behavior of the preferred paths, areas of recirculation and stagnant zones, allowing establishing the non-ideality of the HSSF CW. The simulated time for the peak outlet tracer concentration was 2 days after tracer injection, which was in line with the measured tracer response curve.

To validate the CFD model used in the simulation, concentration data obtained experimentally was compared to those predicted by the model. In order to compare tracer outlet concentrations, graphs were drawn with normalised concentration (tracer concentration measured on time *t*, divided by the integration of all concentration) versus time. The result is shown in Figure 4.55.

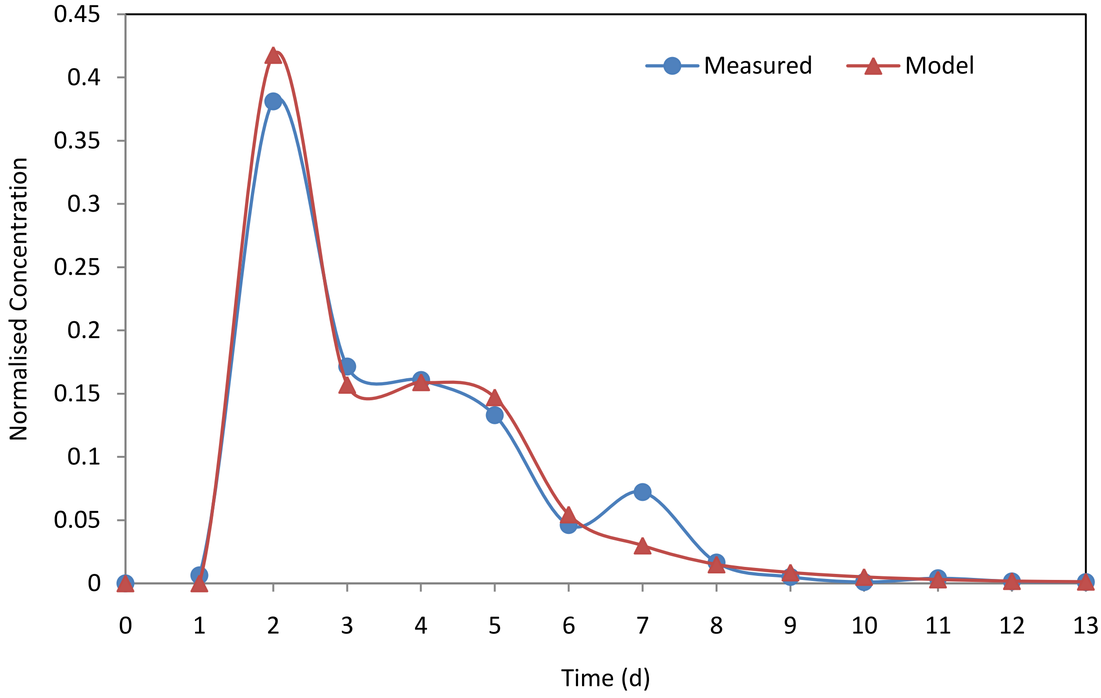


Figure 4.55 Model and measured tracer breakthrough curves

According to the results, it is observed that the modeling of the HSSF CW showed good agreement with the experimental tracer response curve, with a correlation coefficient of 0.99 and thus is considered accurate for further simulations. The very good agreement between the CFD simulated exit tracer concentrations and the measured values, typified by the high correlation coefficient, is an indication the CFD modeling can be a very useful tool for CW evaluation.

4.6.1 Effect of Uniformly Distributed Vegetation and Baffles on Tracer Dynamics

Figure 4.56 shows the two-dimensional velocity fields and streamlines for the HSSF CW with uniformly distributed vegetation, HSSF CW with vertical baffles and HSSF CW with horizontal baffles.

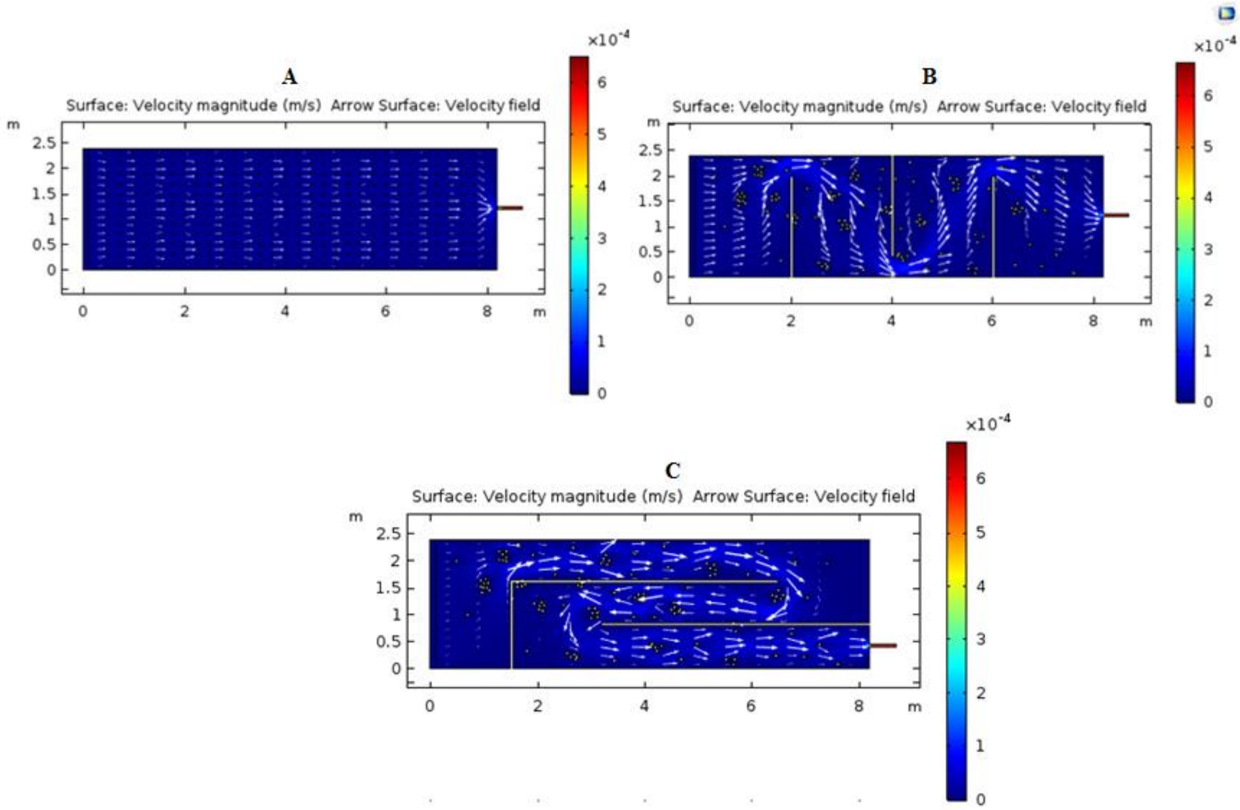


Figure 4.56 Two-dimensional velocity fields and streamlines for all three options evaluated: A) Uniformly distributed vegetation; B) Vertical baffle; C) Horizontal baffles

The spatial distribution of the flow velocity was uniform due to the evenly distributed plant density and this was consistent with the results of previous tracer experiments (Ioannidou and Pearson, 2017). Whereas for the baffled wetlands, regions of higher flow intensities developed due to the presence of the baffles, resulting in significant amount of dead and recirculation zones, which can result to particle sedimentation, affecting the effectiveness of the hydraulic process.

The CFD model simulated two-dimensional evolution of the tracer concentrations for the three hypothetical wetlands are shown in Figures 4.57 and 4.58.

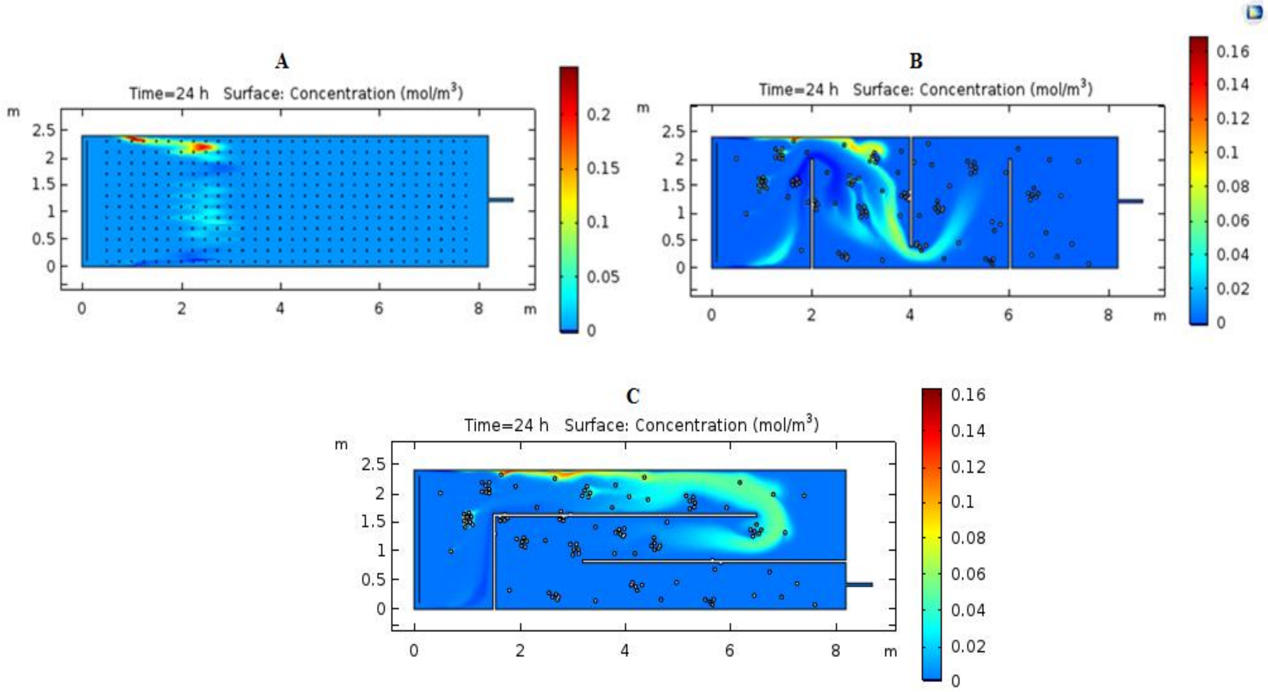


Figure 4.57 Spatial distribution of tracer concentration 24hours after injection

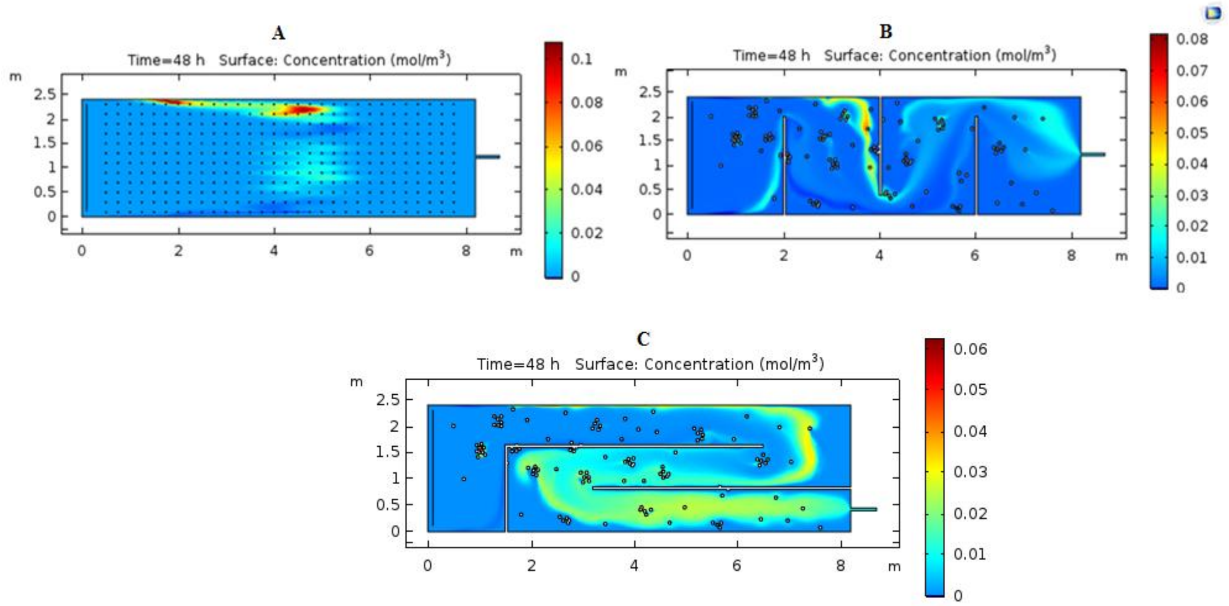


Figure 4.58 Spatial distribution of tracer concentration 48hours after injection

Figure 4.59 shows the CFD predicted tracer response curves for the uniformly distributed vegetation, baffled wetlands as well as the existing PKS HSSF wetland. For the baffled wetlands, the peak tracer outflow occurred at t = 2 days. While the wetland with uniformly distributed vegetation, showed peak tracer outflow at t = 3 days.

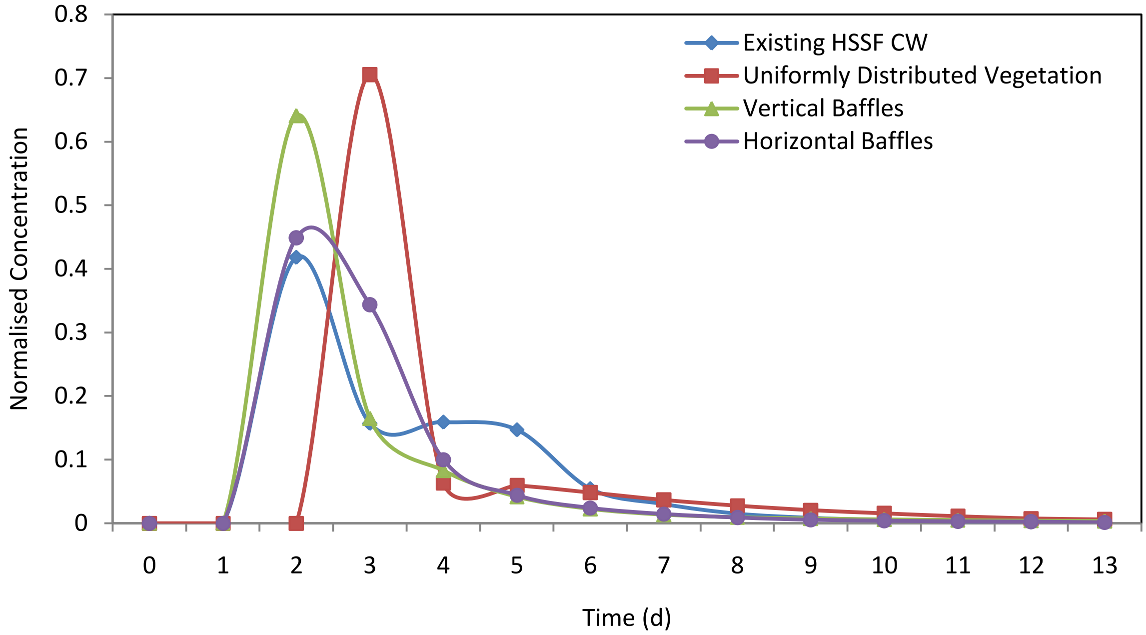


Figure 4.59 Tracer response curves for the different options evaluated

In heterogeneous wetlands, specifically wetlands with substantial variations in vegetation distribution, and other physical characteristics, the physical, chemical, and biological processes will affect the quality of the water differently compared to homogenous wetlands, in which there is little variation in biotic and physical characteristics. Regions of higher flow intensities developed due to the presence of the baffles, resulting in significant amount of dead and recirculation zones, which can result to particle sedimentation, affecting the effectiveness of the hydraulic process.

This is contrary to the submission by Ioannidou and Pearson (2017) in their investigation of short-circuiting in two lagoons, which suggested that baffle curtains retrofitting attenuates short-circuiting by at least 50%, for which they concluded that simple system modification using berms or baffles can improve short-circuiting radically. Also Su et al., (2009) recommended application of obstructions to enhance hydraulic efficiency. Conn and Fiedler (2006) simulated constructed wetlands with baffles and concluded that baffles can markedly increase the hydraulic retention time. Tee et al., (2012) compared a baffled wetland to conventional HSSF CW and reported high NH_4-N removal by the baffled wetland which they attributed to the longer pathway which allowed more contact time of the wastewater with the rhizomes and micro-arobic zones. Therefore, the CFD simulated tracer response curves for the baffled HSSF CW needs further verification with pilot or field scale experiments so as to establish the merits and demerits of placing baffles in such a system. HSSF CW with uniformly distributed vegetation had the most obvious effects on the retention time, with the peak tracer concentration appearing 3 days after injection. Persson *et al*., (1999) showed a similar increase in hydraulic efficiency when a wetland was uniformly vegetated compared with a base case of sparse vegetation near the edges. Because the vegetation to extended across the entire width of the pond, there was no higher velocity flow path in the areas between the inlet and outlet. This result suggests that ensuring a well established and distributed macrophyte population could dramatically improve the poor hydraulic performance in HSSF CWs.

4.6.2 Effect of Wetland Length to Width Ratio on Tracer Dynamics

Three cases of different aspect ratios were investigated. Figure 4.60 shows the tracer response curves for the simulated scenarios. The peak in the RTD for the width to length ratio of 1:2 appeared at t = 2 days, while the peak in the RTD for the width to length ratio of 1:3 and 1:4 appeared at t = 3 days, which showed that increasing the aspect ratio increased the mean residence time and time of peak concentration values of the wetland. This is similar to findings of Zounemat-Kermani et al., (2015) in their simulation of eighty-nine diverse forms of artificial FWS CW with eleven different aspect ratios, as they reported that increasing the aspect ratio has a direct influence on the enhancement of hydraulic efficiency. However, changing the aspect ratio from 1:3 to 1:4 had no significant effect on the time of peak concentration values.

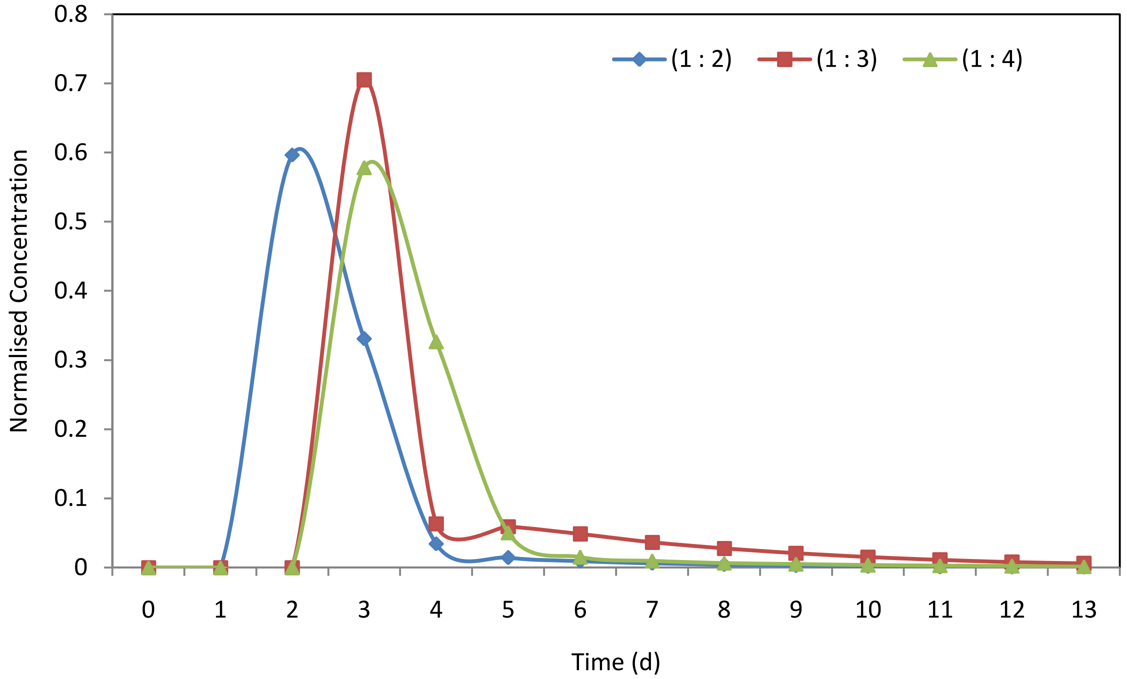


Figure 4.60 Tracer response curves for the different length to width ratios

4.6.3 Evaluation of Alternative HSSF CW Designs

Two alternative HSSF CW designs were investigated and compared to the conventional. The CFD model simulated evolution of the tracer concentrations during the test period are presented in Figures 4.61. It can be seen that the implemented modification (two cells alternative design) generated less re-circulating fluid and short-circuiting flow paths, especially in the second cell, indicating that the flow is more homogeneous; consequently, preferred ways are not favored. So, there is an increase in the residence time that can be verified through the tracer response curves in Figure 4.62.

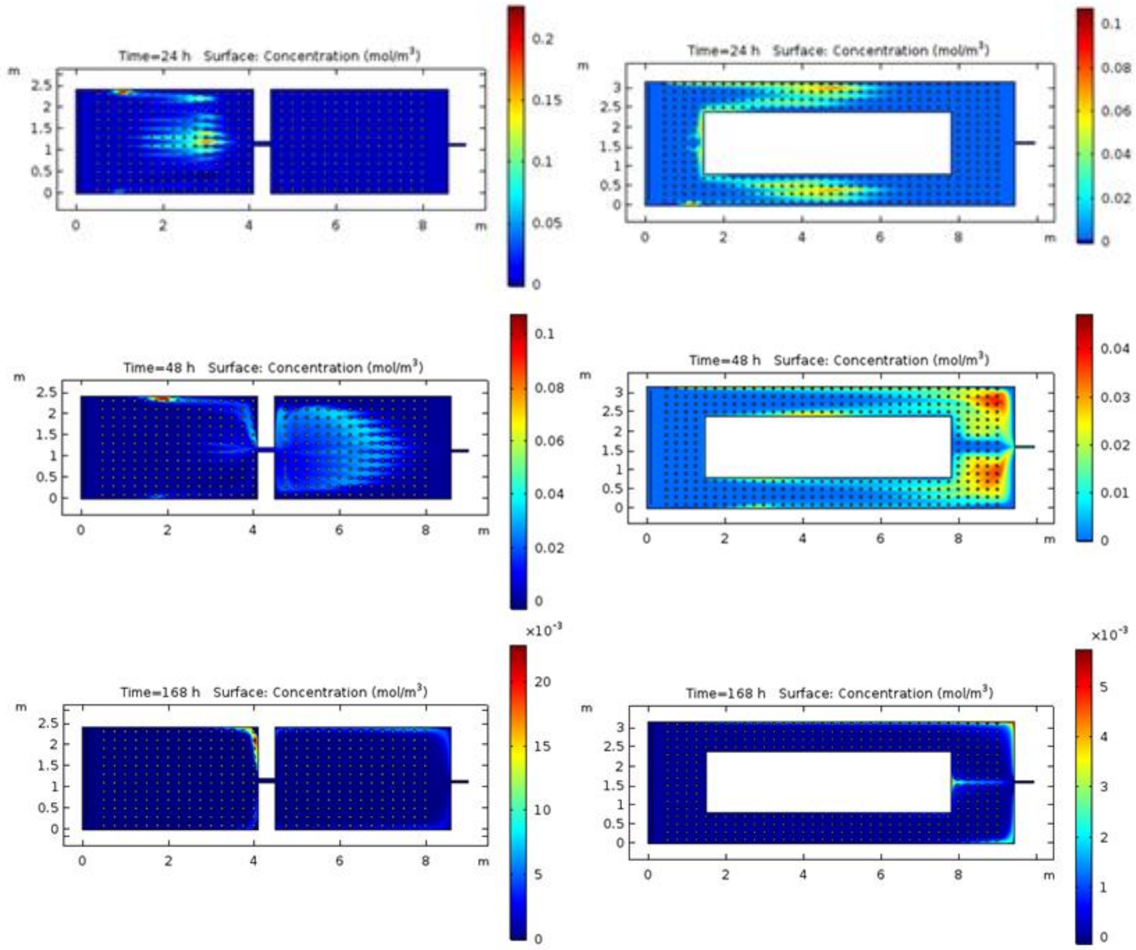


Figure 4.61 Spatial distribution of tracer concentration at different times for alternative designs

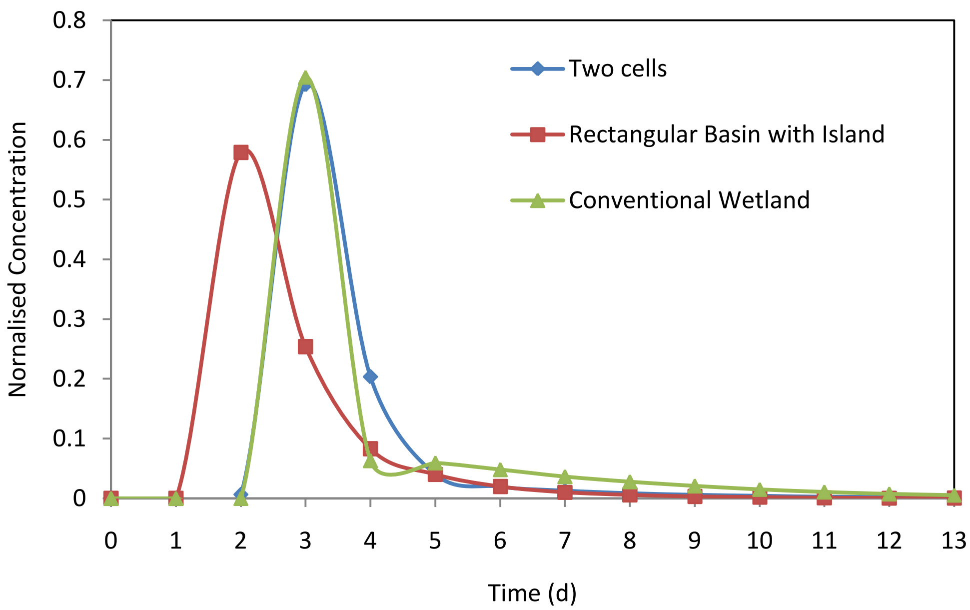


Figure 4.62 Tracer response curves for the different design alternatives

While the two cell system had comparable time of peak concentration values with the conventional wetland, the two cells design is particularly important as it can be designed to allow cleaning and other maintenance activities in one cell while the other cell is still functional, unlike the conventional single cell system. The HSSF CW with island show serious short-circuiting flow paths, with the time of peak concentration value of 2 days, and removing the island significantly improved the hydraulic performance of the wetland.

It is evident that the effects of geometry (i.e. wetland shape and vegetation layout) are fundamental to determining flow fields and the corresponding RTDs. Conventional design methodology for constructed wetlands typically lays more emphasis on factors such as wetland aspect ratio, depth, hydraulic loading rate etc. However, as demonstrated in this study, the optimum design depends on several physical factors such as the shape and length of the system, but a very important consideration is macrophyte density and distribution. The distribution of vegetation within the CFD models of the wetland significantly affected the simulated flow fields and, as a result, their RTDs. The significant short-circuiting observed in the existing system was greatly improved with a uniform vegetation of 25shoots/m^2. Further refinement of the modeling outcome using higher plant densities is recommended.

4.7 Statistical Models for Predicting Effluent Concentrations

MRA was performed to obtain simple regression models for pollutant removal in the PKS-based HSSF CW. Table 4.17 shows the results of the correlation analysis which was conducted prior to the MRA to examine the relationships between an effluent wastewater quality parameter and the influent of other wastewater quality parameters, so as to be able to select suitable parameters for the MRA.

Table 4.17 Correlation matrix of influent and effluent concentrations of water quality parameters.

		Influent							
		BOD	TSS	NH_4^+-N	NO_3^--N	PO_4^{3-}-P	EC	TDS	pH
Effluent	BOD	0.944	0.814	0.376	0.232	0.763	0.816	0.890	-0.411
	TSS	0.863	0.951	0.375	0.215	0.799	0.802	0.887	-0.749
	NH_4^+-N	0.252	0.385	-0.241	0.359	0.230	0.241	0.381	-0.218
	NO_3^--N	0.316	0.226	0.007	-0.029	0.354	0.245	0.331	-0.158
	PO_4^{3-}-P	0.713	0.678	0.054	0.095	0.569	0.581	0.655	-0.423
	EC	0.847	0.859	0.333	0.334	0.868	0.865	0.933	-0.662
	TDS	0.895	0.826	0.295	0.225	0.825	0.802	0.922	-0.679
	PH	-0.482	-0.494	-0.196	-0.256	-0.463	-0.42	-0.549	0.362

The result showed that the effluent concentrations of BOD were strongly related to the influent concentrations of BOD with a correlation coefficient, R, of 0.944; TSS with a correlation coefficient, R, of 0.814; PO_4^{3-} with a correlation coefficient, R, of 0.763; EC with a correlation coefficient, R, of 0.763 and TDS with a correlation coefficient, R, of 0.890. This suggested that the influent loading of these parameters impacts on the final effluent BOD concentration.

The concentration of TSS in the effluent correlated very well with other wastewater quality parameters such as influent concentrations of BOD, TSS, PO_4^{3-}, EC and TDS which produced R values of 0.863, 0.951, 0.799, 0.802 and 0.887 respectively. There was also a strong negative correlation between TSS and pH, with an R value of -0.749. The effluent concentrations of NH_4-N did not correlate strongly with other wastewater quality parameters. So also was the correlation between effluent NO_3-N and other parameters. The result also revealed a strongly relationship between the effluent concentrations of PO_4^{3-} and the influent BOD, TSS and TDS with correlation coefficients of 0.713, 0.678 and 0.655 respectively. The results of the optimization of input variables for predicting final effluent concentrations for BOD, TSS, NH_4-N, NO_3-N and PO_4^{3-} are presented in Table 4.18.

Table 4.18 Optimal input variables combination for predicting final effluent concentrations.

	Input Variables	R^2	Sig F	P Values			S
BOD	1	0.891	0.000	0.000			6.88
	1 + 3	0.898	0.000	0.000	0.226		6.80
	1 + 2 + 3	0.900	0.000	0.000	0.526	0.181	6.89
TSS	2	0.905	0.000	0.000			5.55
	2 + 3	0.919	0.000	0.000	0.064		5.24
	1 + 2 + 3	0.919	0.000	0.885	0.000	0.170	5.34
NH_4-N	2	0.149	0.057	0.057			3.39
	2 + 4	0.232	0.055	0.137	0.100		3.29
	2 + 3 + 4	0.233	0.127	0.507	0.851	0.163	3.36
NO_3-N	5	0.125	0.083	0.083			3.08
	1 + 5	0.127	0.224	0.413	0.818		3.14
	1 + 3 + 5	0.127	0.403	0.901	0.951	0.537	3.21
PO_4^{3-}	1	0.509	0.000	0.000			1.05
	1 + 2	0.523	0.000	0.421	0.100		1.05
	1 + 2 + 3	0.524	0.001	0.163	0.427	0.847	1.10

1 - inf BOD; 2 - inf TSS; 3 - inf TDS; 4 - inf NO_3^--N; 5 - inf PO_4^{3-}

From the results presented in Table 4.18, it can be seen that for BOD prediction, the combination of predictor variables that returned the best R^2 of 0.898 and the lowest standard error of the estimation (S) of 6.80 was that of influent BOD and TDS. The P-values showed that only influent BOD had a significant effect on the output BOD. This was also evident from the fact that the model constructed for predicting effluent BOD concentrations using just one predictor variable (influent BOD), gave an R^2 of 0.891. This suggests that influent BOD concentration can also be solely used to predict the effluent BOD concentration.

For TSS, the combination of predictor variables that returned the best R^2 of 0.919 and the lowest standard error of the estimation (S) of 5.24 was that of influent TSS and TDS. The P-values also revealed that only influent TSS had a significant effect on the output TSS. The equally high R^2 value obtained for influent TSS showed that the one predictor variable was efficient. The most efficient model developed for predicting final effluent concentrations of NH_4-N consisted of two input variables TSS and NO_3-N. The model gave an R^2 value 0.232. Both predictor variables of influent TSS and NO_3-N recorded P-values that were not low enough to be considered significant.

The model developed using a single predictor variable PO_4^{3-} was adjudged to be the best in predicting final effluent concentrations of NO_3-N. However, the model returned a very low R^2 value of 0.125 and a standard error of the estimation (S) of 3.08. The model also had an insignificant statistical relationship. The model developed using a combination of BOD and TSS was adjudged to be the best in predicting final PO_4^{3-} effluent concentrations. It was not as strong as models developed for predicting other dependant variables such as BOD as can be seen from the R^2 of 0.523.

From Table 4.18 it can be seen that for NH_4-N, NO_3-N and PO_4^{3-}, most R^2 values are very low and P-values very high, compared to the values obtained for BOD and TSS. This is attributed to two likely possibilities, first is the fact that straight line models may not help in predicting nutrient concentrations. Secondly, the treatment processes cannot be adequately described by MRA. The regression models adjudged to be the best in predicting the final effluent concentrations for the selected pollutants are presented in Table 4.19.

Table 4.19 Optimum models for predicting final effluent concentrations from multiple regression analysis.

BOD	0.109 (Inf BOD)* + 0.011(Inf TDS)	+ 14.119
TSS	0.153 (Inf TSS)* + 0.012 (Inf TDS)	- 7.823
NH_4^+-N	0.209 (Inf NO_3^--N) + 0.014 (Inf TSS)	+ 24.013
NO_3^--N	0.540 (Inf PO_4^{3-}-P)	+ 14.862
PO_4^{3-}-P	0.004 (Inf TDS) + 0.005 (Inf BOD)	+ 2.403

*= $p \leq 0.05$

The statistical models developed for predicting final effluent concentrations of selected key wastewater quality parameters using MRA were found to be promising. The MRA model for predicting final effluent BOD and TSS concentrations was exceptionally good and this was reiterated by the R^2 values of each predictor variable. However, in the case of nitrogen the best MRA model for predicting final effluent NH_4-N and NO_3-N concentrations gave low R^2 values and this suggested that the models were not strong in predicting final effluent concentrations. Statistical models developed from multiple regression analysis can be considered a simple and useful tool to provide information on the overall performance of wetlands. However, statistical models are typically considered valid only for the range of data used to model them (Stone *et al*., 2002), thus caution must be exercised in their utilization.

4.7 Interview Results

The structured interview was carried out to provide an insight on perception of CWs by the slaughterhouse operators and users. Although the sample size of the survey was small, it provided very useful information. At the time of the study, none of the respondents had a prior knowledge of the possibilities of using CW to treat wastewater and had never seen the technology before. 65% of the respondents expressed scepticism about the viability of the system, while 85% expressed happiness at the improved sanitation conditions during the study period. 15% of the respondents were indifferent, stating that they cared less about the environmental conditions, as long as their daily income was not affected. All the respondents expressed concerns as regards financing the installation and maintenance of a field-scale system, as well as the running costs compared to septic tanks.

From the comments of the users and members of the general public and from the researchers' view, it is evident that the wetland technology for wastewater treatment is a new concept and is not yet understood by a wider public. Although great interest was expressed, more public awareness and demonstration is essential not only for the small systems but also for bigger ones.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

From the findings and the foregoing discussions, the following conclusions are derivable from the study:

1. The concentration of most slaughterhouse wastewater physicochemical parameters was very high, with most values exceeding the set standards for effluent discharge into the environment. However, the study also revealed that effluents from the slaughterhouses can be effectively treated by biological means. Another key conclusion of the study is the fact that almost all the major slaughterhouses in Anambra State lacked the infrastructure to support the production of safe and wholesome meat and meat products. Solid and liquid waste management at the slaughterhouses were poor. This corroborated the assertion by Adeyemo (2002) that waste generated as a result of abattoir operation in Nigeria is a source of embarrassment to the general public. Slaughterhouses in Anambra State urgently need effluent treatment facilities to reduce the health hazard their effluent pose on the slaughterhouse users and users of the receiving environment.
2. *Thalia Geniculata* was found to be an effective plant species for HSSF CW treating slaughterhouse effluent. It also survives and proliferates in minimally washed PKS, although with negative effects on the height and shoot generation rate. All the evaluated species showed comparable pollutant removal capacity. Therefore, it is suggested that a mixture of macrophyte species available in the area might provide the best long-term option. The use of PKS as a wetland substrate revealed satisfactory organic matter, nutrient and suspended solids removal, comparable to the levels achieved with the conventional gravel substrate. Therefore it is recommended as an alternative substrate material for constructed wetlands.

3. Designing HSSF CWs using appropriate model constants specific to the wastewater and substrate of interest and obtained at the particular environment where the system will be used, rather than relying on values determined for other types of wastewater and substrates, in other regions of the world, would make treating wastewater with CWs more effective and efficient.
4. PKS based field-scale HSSF CW has the capacity to significantly reduce organic pollutants as well as suspended solids in slaughterhouse effluent. Under the present design and operating conditions, nutrients removal capacity of this wetland system is very low. However, the results are comparable to the removal rates using other more expensive substrate materials, thereby justifying the use of PKS as a substrate material. Although the effluent quality did not meet the discharge standard for most parameters, very significant reductions were achieved with the system, which indicates it could provide a better alternative to conventional wastewater treatment systems such as activated sludge.
5. PKS based field-scale CW general had a good hydraulic efficiency, with a moderate amount of dead zones. CFD modelling of the system hydrodynamics showed good agreement with the experimental results, allowing for the verification of its hydrodynamic behavior. Based on the modelling approach, the current design configuration appears to be sufficient for producing functional wetlands, but emphasis should be laid on the distribution of vegetation in the system. CFD modelling is an effective tool for simulation and optimization during the design phase and can also reduce the operating and maintenance costs.

At the end of this study, the experiments, tests, analysis and results have, as expected, qualified and quantified the hydraulic efficiency of the PKS-based field-scale HSSF CW which was investigated, as well as characterized the water quality parameters removal processes and efficiencies. Generally, it can be concluded that the treatment performance of the system was very encouraging in promoting the use of constructed wetlands as an alternative wastewater treatment system for protecting sensitive water bodies that receive partially treated or untreated slaughterhouse effluents. In addition, for a developing country like Nigeria, that has limited resources for the construction and operation of conventional treatment plants, PKS-based CWs is the most economical solutions. Overall, the data presented in this thesis provide excellent base information which could be widely used for further research on PKS-based CWs under tropical conditions.

5.2 Recommendations

Within the limit of experience gained in the cause of this study, the following recommendations are made:

1. Considering this is one of the first attempts to evaluate PKS as a CW substrate on a field-scale, the results were deemed successful, but also provide a focus for future research. The first priority for such research would be to evaluate the long term performance of the substrate, with a view to determining the magnitude of its lifespan in a CW.
2. Long term (5 to 10 years) treatment performance of the CW is important. The age range (< 1year) of the studied systems was relatively small and primarily consisted of "young" plants. Therefore, analyzing several years of data to develop seasonal trends in performance is important to accurately establish treatment capabilities of PKS and to develop accurate removal rate constants that can be used for system design.
3. It has also been demonstrated that single tracer tests should not be considered as providing absolute information on the hydraulic behaviour of a CW. The information provided by the tracer test in this study should be considered as a snapshot of a system that is highly sensitive to spatial and temporal variations. Therefore, replication may provide significantly different results.

4. Finally, experimentation on different systems, including vertical flow and hybrid wetlands, using PKS as substrate and other available macrophyte should be considered. Designs reducing the area needed for wastewater treatment would undeniably allow broader adoption of CWs. However, utilization of local materials as much as possible should be the driving force.

5.3 Contribution to Knowledge

This research is the first among any know literature to evaluate the performance of a field-scale CW with PKS as a substrate, and also evaluate its hydrodynamic behavior, which is extremely important for future design, operation, and maintenance, was evaluated. The contributions of this study to the interdisciplinary field of CW are:

1. This is a pioneer study, and thus will serve as a foundation for further studies regarding implementation of PKS based field-scale wetlands for wastewater treatment.
2. The study has generated design model constants that are specific to palm kernel shell based horizontal subsurface flow constructed wetland that are lacking in wetland literature.
3. The research has also provided rigorous field data and information on the treatment performance of palm kernel shell based horizontal subsurface flow constructed wetland.
4. A computational fluid dynamic model which can be used for the design and evaluation of horizontal subsurface flow wetlands was developed and validated.

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Appendix A

Publications and Presentation from the PhD Research

The following publications are as a result of this research:

Okoye, N. M., Madubuike, C. N., Nwuba, U. I. and Orakwe, L. C. (2018). Growth and treatment performance of three macrophytes in a pilot-scale horizontal flow constructed wetland for slaughterhouse wastewater. *Archives of Current Research International*.14(1): 1-7; Article no.ACRI.39333. ISSN: 2454-7077

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Appendix B

Calculated Constructed Wetland Areas

B.1 Calculated area for BOD removal

Month	Flowrate	Temp	Θ	k_{20}	K	Area(m^2)	C*
Jan	0.34	27.4	1	0.604167	0.604167	7.756942	23
Feb	0.38	29.6	1	0.604167	0.604167	8.669523	23
Mar	0.47	30.2	1	0.604167	0.604167	10.72283	23
Apr	0.72	29.3	1	0.604167	0.604167	16.42646	23
May	1.02	28.1	1	0.604167	0.604167	23.27083	23
Jun	1.05	27.5	1	0.604167	0.604167	23.95526	23
Jul	1.02	26.7	1	0.604167	0.604167	23.27083	23
Aug	1.04	26.4	1	0.604167	0.604167	23.72712	23
Sep	1.28	26.8	1	0.604167	0.604167	29.2026	23
Oct	0.96	27	1	0.604167	0.604167	21.90195	23
Nov	0.41	28.6	1	0.604167	0.604167	9.353959	23
Dec	0.29	27.4	1	0.604167	0.604167	6.616215	23
Average.	0.74833333	27.91667	1	0.604167	0.604167	17.07288	23

B.2 Calculated area for TSS removal

Month	Flowrate	Temp	Θ	k_{20}	K	Area(m^2)	C*
Jan	0.34	27.4	1.092801	0.623646	1.202662	6.58629	25.6
Feb	0.38	29.6	1.092801	0.623646	1.461956	6.055567	25.6
Mar	0.47	30.2	1.092801	0.623646	1.54191	7.101405	25.6
Apr	0.72	29.3	1.092801	0.623646	1.423548	11.78328	25.6
May	1.02	28.1	1.092801	0.623646	1.279742	18.56877	25.6
Jun	1.05	27.5	1.092801	0.623646	1.213382	20.16031	25.6
Jul	1.02	26.7	1.092801	0.623646	1.130225	21.02524	25.6
Aug	1.04	26.4	1.092801	0.623646	1.100531	22.0159	25.6
Sep	1.28	26.8	1.092801	0.623646	1.140299	26.1515	25.6
Oct	0.96	27	1.092801	0.623646	1.160719	19.26858	25.6
Nov	0.41	28.6	1.092801	0.623646	1.337806	7.139968	25.6
Dec	0.29	27.4	1.092801	0.623646	1.202662	5.617718	25.6
Average	0.748333	27.91667	1.092801	0.623646	1.266287	14.28954	25.6

B.3 Calculated area for NH_4-N removal

Month	Flowrate	Temp	Θ	k_{20}	K	Area(m^2)	C*
Jan	0.34	27.4	1.050477	0.278271	0.400618	7.892811	0
Feb	0.38	29.6	1.050477	0.278271	0.446458	7.915632	0
Mar	0.47	30.2	1.050477	0.278271	0.459847	9.505343	0
Apr	0.72	29.3	1.050477	0.278271	0.439911	15.22126	0
May	1.02	28.1	1.050477	0.278271	0.414668	22.87611	0
Jun	1.05	27.5	1.050477	0.278271	0.402595	24.25512	0
Jul	1.02	26.7	1.050477	0.278271	0.387043	24.50889	0
Aug	1.04	26.4	1.050477	0.278271	0.381367	25.36138	0
Sep	1.28	26.8	1.050477	0.278271	0.388954	30.60517	0
Oct	0.96	27	1.050477	0.278271	0.392804	22.72891	0
Nov	0.41	28.6	1.050477	0.278271	0.425005	8.971655	0
Dec	0.29	27.4	1.050477	0.278271	0.400618	6.732103	0
Average	0.748333	27.91667	1.050477	0.278271	0.411657	17.21453	0

B.4 Calculated area for NO_3-N removal

Month	Flowrate	Temp	Θ	k_{20}	K	Area(m^2)	C*
Jan	0.34	27.4	1.015615	0.323541	0.362849	3.607564	0.36
Feb	0.38	29.6	1.015615	0.323541	0.375431	3.896856	0.36
Mar	0.47	30.2	1.015615	0.323541	0.378938	4.775194	0.36
Apr	0.72	29.3	1.015615	0.323541	0.37369	7.417917	0.36
May	1.02	28.1	1.015615	0.323541	0.366806	10.70594	0.36
Jun	1.05	27.5	1.015615	0.323541	0.363411	11.12376	0.36
Jul	1.02	26.7	1.015615	0.323541	0.358934	10.94072	0.36
Aug	1.04	26.4	1.015615	0.323541	0.35727	11.20722	0.36
Sep	1.28	26.8	1.015615	0.323541	0.359491	13.70827	0.36
Oct	0.96	27	1.015615	0.323541	0.360607	10.24939	0.36
Nov	0.41	28.6	1.015615	0.323541	0.369659	4.270157	0.36
Dec	0.29	27.4	1.015615	0.323541	0.362849	3.07704	0.36
Average	0.748333	27.91667	1.015615	0.323541	0.365828	7.915002	0.36

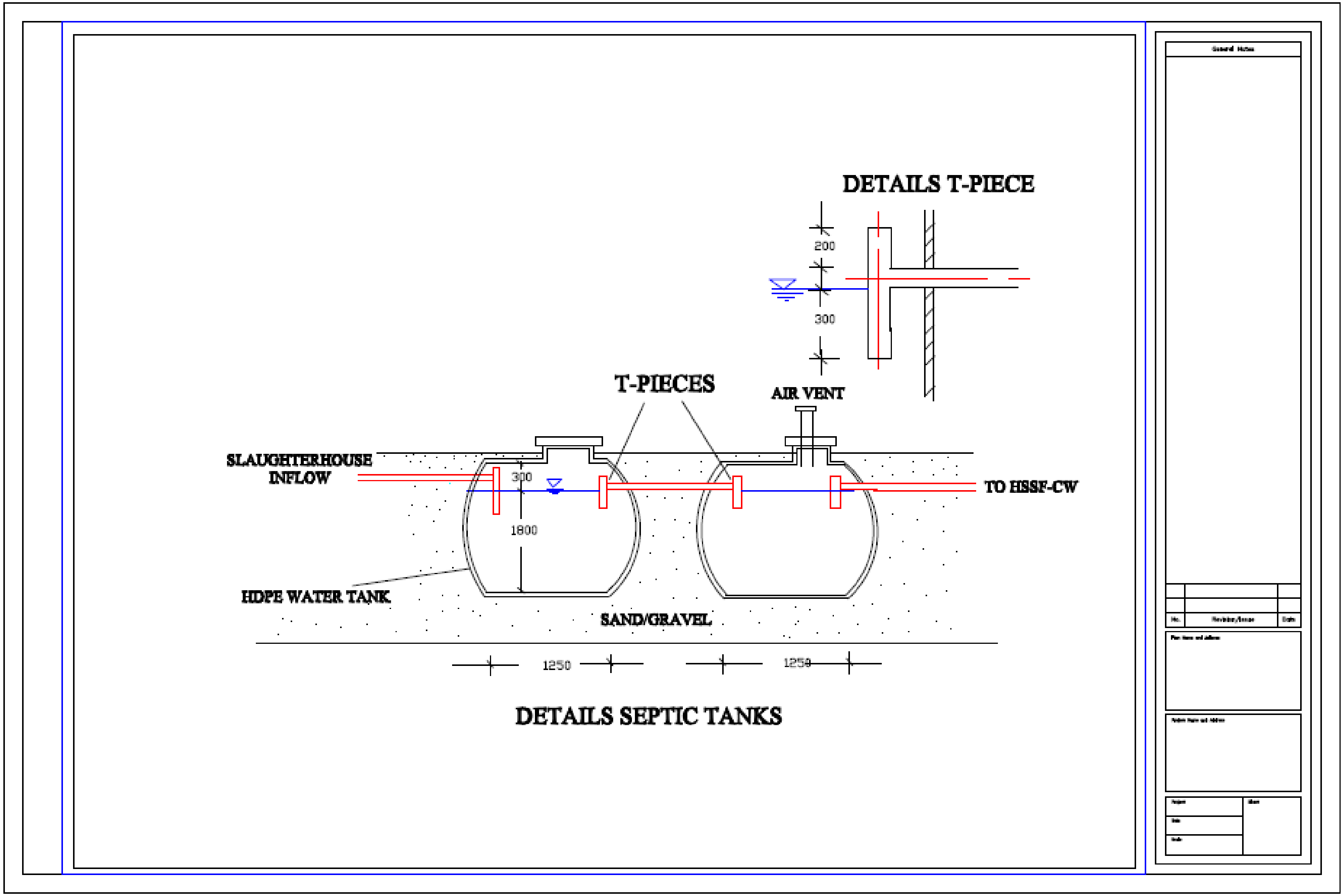
B.5 Calculated area for PO_4^{3-} removal

Month	Flowrate	Temp	Θ	k_{20}	K	Area(m^2)	C*
Jan	0.34	27.4	0.953686	0.306455	0.215758	5.279072	0.42
Feb	0.38	29.6	0.953686	0.306455	0.194383	6.548935	0.42
Mar	0.47	30.2	0.953686	0.306455	0.18893	8.333774	0.42
Apr	0.72	29.3	0.953686	0.306455	0.197168	12.23323	0.42
May	1.02	28.1	0.953686	0.306455	0.208713	16.37175	0.42
Jun	1.05	27.5	0.953686	0.306455	0.214737	16.38051	0.42
Jul	1.02	26.7	0.953686	0.306455	0.22304	15.32013	0.42
Aug	1.04	26.4	0.953686	0.306455	0.226236	15.39988	0.42
Sep	1.28	26.8	0.953686	0.306455	0.221985	19.31665	0.42
Oct	0.96	27	0.953686	0.306455	0.219889	14.62554	0.42
Nov	0.41	28.6	0.953686	0.306455	0.203823	6.738701	0.42
Dec	0.29	27.4	0.953686	0.306455	0.215758	4.502738	0.42
Average	0.748333	27.91667	0.953686	0.306455	0.210868	11.75424	0.42

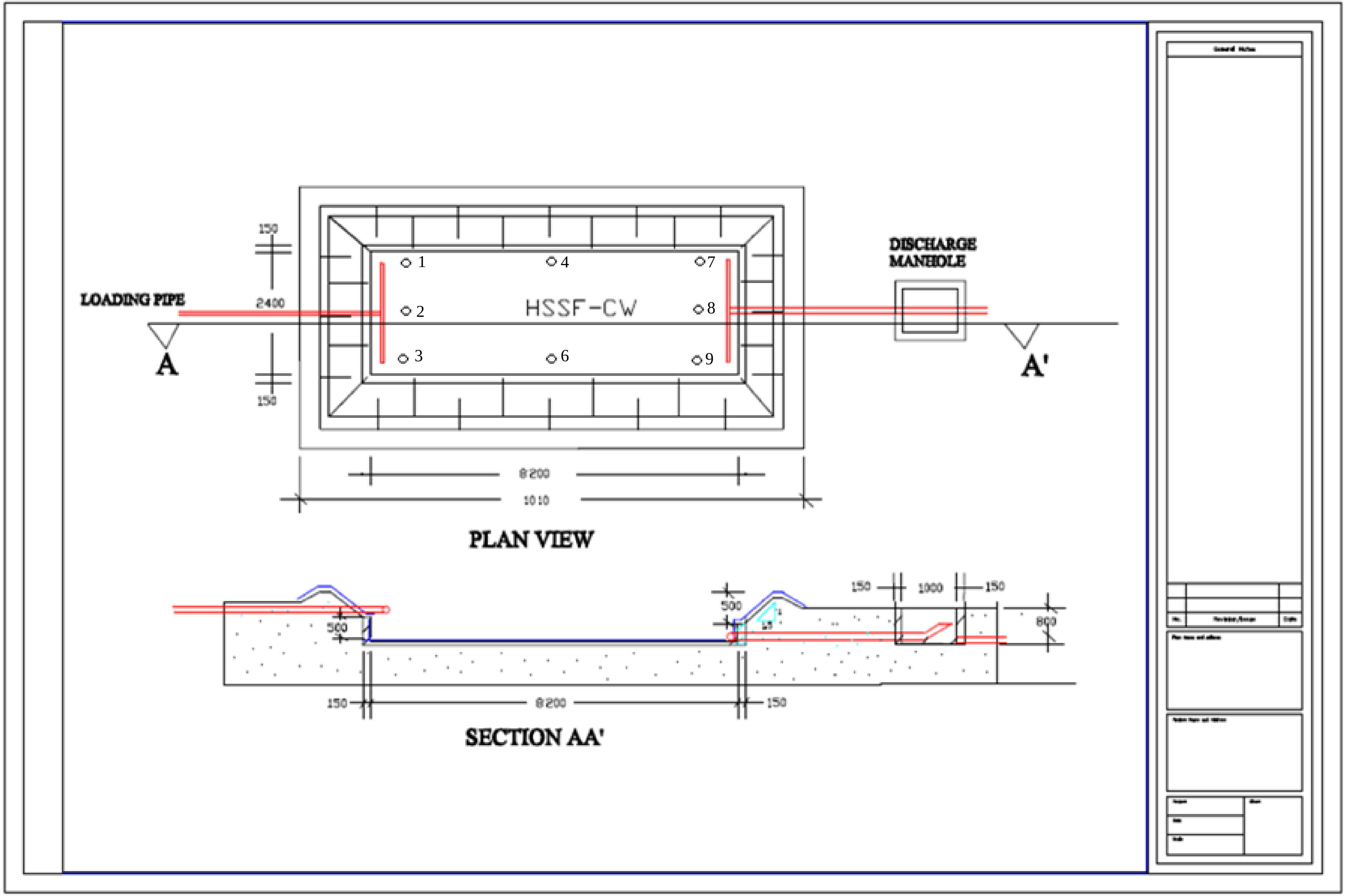
Appendix C

Design drawings for the bioremediation system

C.1 Septic tank



C.2 Horizontal subsurface flow constructed wetland



Appendix D

Construction Steps for the Onsite Bioremediation System

D.1 Excavation for septic tank and constructed wetland

D.2 Installation of the two chamber experimental septic tank

D.3 Connection of the slaughter slab to the septic tank

D.4 Installation of sanitary tee pipe fittings


D.5 Installation of inlet distribution pipe


D.6 Compaction and rendering with a mixture of sand, cement and water seal


D.7 Lining with a high density polyethylene sheet and filling with PKS

D.8 Installation of outlet drain pipe and excavation of the discharge manhole

D.9 Planting of macrophyte and waterproof test

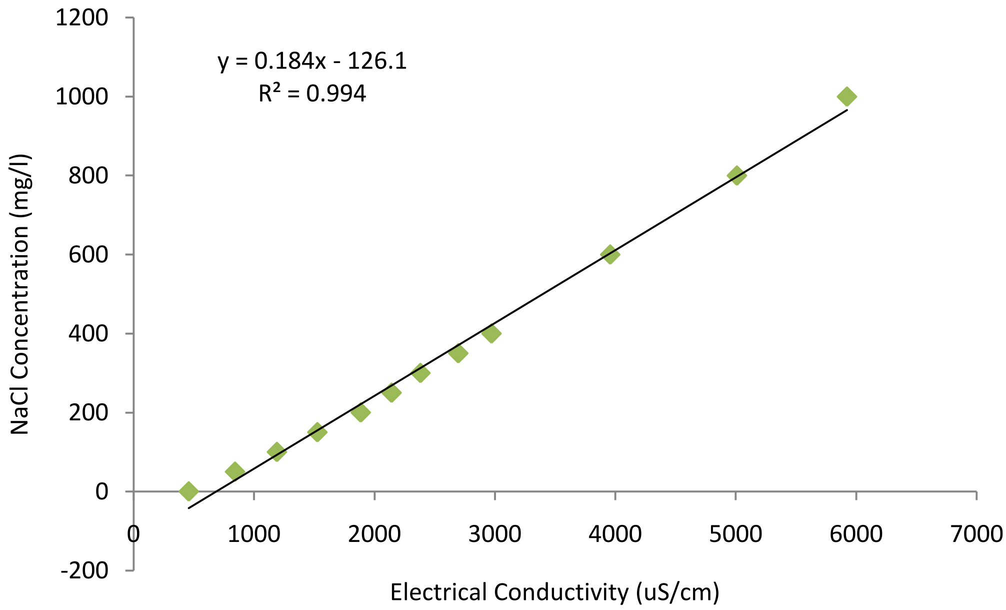
Appendix E

Tracer Calibration

Soduim Chloride Dilutions

NaCl Tracer Concentration (mg/l)	Electrical Conductivity (uS/cm)
0	458
50	842
100	1190
150	1526
200	1886
250	2142
300	2382
350	2696
400	2972
600	3958
800	5010
1000	5924

NaCl calibration graph



Appendix F

Statistical Analysis

BOD removal efficiency of four pilot wetland cells

Control	*Typha Latifolia*	*Thalia Geniculata*	*Colocasia Esculenta*
67.07	85.15	89.98	67.17
59.16	81.27	80.06	72.01
57.21	81.07	74.05	69.80
60.23	87.06	70.82	74.14
70.05	87.24	80.16	79.14
69.20	82.03	75.04	76.86
65.41	88.02	79.15	70.80
49.11	69.16	67.48	65.31
67.12	70.19	67.06	69.04
70.13	79.00	68.25	60.57
55.12	79.00	69.53	67.37
42.05	72.41	73.68	64.61
50.58	73.98	77.59	68.40
45.52	73.05	82.99	72.07

F.1 Data Normality Test

Case Processing Summary

	Cell	Cases					
		Valid		Missing		Total	
		N	Percent	N	Percent	N	Percent
BOD	Control	14	100.0%	0	0.0%	14	100.0%
	Typha Latifolia	14	100.0%	0	0.0%	14	100.0%
	Thalia Geniculata	14	100.0%	0	0.0%	14	100.0%
	Colocasia Esculenta	14	100.0%	0	0.0%	14	100.0%

Tests of Normality

	Cell	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
		Statistic	Df	Sig.	Statistic	df	Sig.
BOD	Control	.173	14	.200*	.918	14	.208
	Typha Latifolia	.146	14	.200*	.927	14	.281
	Thalia Geniculata	.112	14	.200*	.945	14	.480
	Colocasia Esculenta	.108	14	.200*	.989	14	.999

*. This is a lower bound of the true significance.

a. Lilliefors Significance Correction

All the Sig. values for the Shapiro-Wilk test were greater than 0.05, thus the data were all normally distributed. Data transformation was not carried out.

F.2 One-way ANOVA (P<0.05, Data is statistically different)

Descriptives

BOD

	N	Mean	Std. Deviation	Std. Error	95% Confidence Interval for Mean	
					Lower Bound	Upper Bound
Control	14	59.1434	9.52388	2.54536	53.6445	64.6423
Typha Latifolia	14	79.1927	6.49513	1.73590	75.4425	82.9429
Thalia Geniculata	14	75.4231	6.66263	1.78066	71.5762	79.2700
Colocasia Esculenta	14	69.8111	4.91702	1.31413	66.9721	72.6501
Total	56	70.8926	10.28662	1.37461	68.1378	73.6473

Descriptives

BOD

	Minimum	Maximum
Control	42.05	70.13
Typha Latifolia	69.17	88.02
Thalia Geniculata	67.07	89.99
Colocasia Esculenta	60.58	79.14
Total	42.05	89.99

ANOVA

BOD

	Sum of Squares	Df	Mean Square	F	Sig.
Between Groups	3200.831	3	1066.944	21.184	.000
Within Groups	2618.964	52	50.365		
Total	5819.795	55			

Post Hoc Tests

Multiple Comparisons

Dependent Variable: BOD

Tukey HSD

(I) Cell	(J) Cell	Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval Lower Bound
Control	Typha Latifolia	-20.04932*	2.68234	.000	-27.1685
	Thalia Geniculata	-16.27971*	2.68234	.000	-23.3989
	Colocasia Esculenta	-10.66775*	2.68234	.001	-17.7869
Typha Latifolia	Control	20.04932*	2.68234	.000	12.9301
	Thalia Geniculata	3.76962	2.68234	.502	-3.3496
	Colocasia Esculenta	9.38157*	2.68234	.005	2.2624
Thalia Geniculata	Control	16.27971*	2.68234	.000	9.1605
	Typha Latifolia	-3.76962	2.68234	.502	-10.8888
	Colocasia Esculenta	5.61196	2.68234	.169	-1.5072
Colocasia Esculenta	Control	10.66775*	2.68234	.001	3.5486
	Typha Latifolia	-9.38157*	2.68234	.005	-16.5008
	Thalia Geniculata	-5.61196	2.68234	.169	-12.7312

Appendix H

Tracer Test Data

H.1 Computational procedure for the tracer RTD

t	NaCl Tracer Concentration (mg/l)	C(t)dt	t C(t) dt	(t-τ)^2	(t-τ)^2 C(t) dt
0	0	0	0	0	0
1	1.964	1.964	1.964	6.949033	13.6479
2	118.068	118.068	236.136	2.676829	316.0479
3	53.116	53.116	159.348	0.404626	21.49209
4	49.804	49.804	199.216	0.132422	6.595137
5	41.156	41.156	205.78	1.860218	76.55913
6	14.292	14.292	85.752	5.588014	79.8639
7	22.388	22.388	156.716	11.31581	253.3384
8	5.092	5.092	40.736	19.04361	96.97005
9	1.596	1.596	14.364	28.7714	45.91916
10	0.308	0.308	3.08	40.4992	12.47375
11	1.228	1.228	13.508	54.227	66.59075
12	0.492	0.492	5.904	69.95479	34.41776
13	0.308	0.308	4.004	87.68259	27.00624
		309.812	1126.508		1050.922

$\tau = \sum t\ C(t)\ dt \ / \sum C(t)dt = 1126.508/309.81 = 3.64\text{days}$

$\sigma^2 = \sum(t-\tau)^2\ C(t)\ dt \ / \sum C(t)dt = 1050.92/309.81 = 3.39\text{days}^2$

Appendix I

Picture Gallary

Young Cattail shoots stored in water before planting

Slaughter slab of the Agulu slaughterhouse

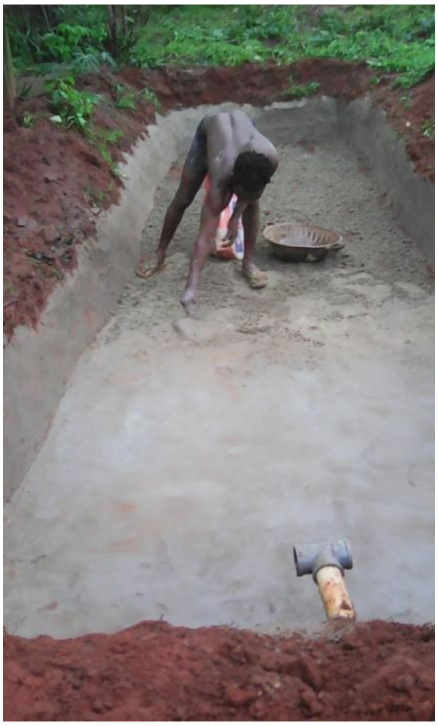
Agulu slaughterhouse lairage

Samples collected from pilot wetland cells

Pilot wetlands

Refilling the reservoir

Calibration of pilot wetland cells

Sand and cement rendering of the HSSF wetland


Wastewater sample collection at the studied slaughterhouses

Wastewater filtration for TSS determination

DO meter and magnetic stirrer for BOD Measurement

Nutrient analysis using the Hach test kits.

Measurement of aggregate thickness using a vernier caliper