EVALUATING THE HYDRAULIC PERFORMANCE OF VERTICAL FLOW CONSTRUCTED WETLAND SYSTEM IN EFFLUENT POLISHING

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MAINA CAROLINE WANGECI

A Thesis Submitted to the Graduate School in Partial Fulfilment for the requirement of the Master of Science Degree in Water Resources and Environmental Management of Egerton University

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DECLARATION AND RECOMMENDATION

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I declare that this Thesis is my original work and has not been submitted to any other institution known to me for award of any other degree.

Signature.....Date 10/6/2011

Maina, C. W.

Reg. No: BM13/2248/08

RECOMMENDATION

This Thesis has been submitted for examination with our approval as university supervisors.

Supervisors

Signature.....Date 10/06/2011

Dr. B. M. Mutua

Department of Agricultural Engineering, Egerton University

Signature.....Date 10/6/2011

Dr. S.O. Oduor

Department of Biological Sciences, Egerton University

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DEDICATION

Dedicated to my parents Joseph and Esther Maina, brothers and sisters whose inspiration and support was a major motivation in my academic endeavours.

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Glory and honour be to the almighty God who has been the source of my strength throughout my life and studies.

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ABSTRACT

The discharge of untreated wastewater or partially treated effluent and runoff from agricultural fields into water bodies is a major source of surface water pollution worldwide. To mitigate this pollution problem, wastewater treatment using wastewater stabilisation ponds and constructed wetlands have been promoted. Such wastewater treatment systems have their performance strongly dependent on their hydraulics which if not properly considered during design or operation, may result into partially treated effluent being discharged into water bodies. Disposal of such partially treated wastewater into water bodies in large quantities lowers their self purification ability. This study evaluated the hydraulic performance of a vertical flow constructed wetland system under varying hydraulic loading regimes. The study also simulated treatment of wastewater from Constructed Wetlands (CWs) using PREWet model. Influent and effluent samples from the constructed wetland were collected and analysed for physical, chemical and biological parameters of importance to water quality based on recommended standard laboratory methods. The data collected was useful in determining the treatment efficiency of the wetland. A statistical evaluation, ANOVA using SPSS programme was used to analyse the performance of the constructed wetland system under varying hydraulic loading rates. The results showed that to adequately polish effluent from wastewater stabilisation ponds, a hydraulic retention time of 2 days was required. Phosphorous reduction for the different hydraulic loading rates ranged between 91.59% and 47.16% for the lowest and highest loading rates applied respectively. However, Ammonium nitrogen reduction was not significantly affected by the different hydraulic loading rates, since the reduction ranged between 97% and 92%. The modelled parameters included Total Phosphorous, Biological Oxygen Demand, Total Suspended Solids and coliform whose coefficient of correlation ranged between 0.90 and 0.99. The PREWet model was found to be effective in simulation of wastewater polishing by constructed wetlands therefore suggesting that the model can act as an important tool to planners and water resources managers. The results from this study are useful in enhancing use of decentralized wastewater treatment systems.

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TABLE OF CONTENTS

DECLARATION AND RECOMMENDATION ii
COPYRIGHT iii
DEDICATION iv
ACKNOWLEDGEMENT v
ABSTRACT vi
TABLE OF CONTENTS vii
LIST OF FIGURES x
LIST OF TABLES xi
LIST OF PLATES xii
ACRONYMS xiii
DEFINITION OF TERMS xiv
CHAPTER ONE 1
INTRODUCTION 1
1.1 Background 1
1.2 Statement of the Problem 2
1.3 Objectives 3
1.3.1 Specific Objectives 3
1.3.2 Research Questions 3
1.4 Justification 3
CHAPTER TWO 5
LITERATURE REVIEW 5
2.1 Introduction 5
2.2 Eutrophication 5
2.3 Wastewater treatment 6
2.3.1 Wastewater treatment in developing countries 6
2.3.2 Wastewater stabilisation ponds (WSPs) 7
2.4 Effluent polishing and disinfection 7
2.4.1 Chlorination method 8
2.4.2 Solar disinfection (SODIS) 8
2.5 Wetlands 9
2.6 Wetlands plants 9
2.7 Constructed wetlands (CWs) 10
2.7.1 Surface Flow (SF) 11

2.7.2 Subsurface Flow (SSF) 11
2.8 Design of Subsurface flow constructed wetland 12
2.9 Pollutant removal in CWs 14
2.10 Self purification ability of a river 15
2.11 Wastewater treatment models 15
2.11.1 Wetland Water Quality Model (WWQM) 16
2.11.2 Constructed Wetland 2 - Dimensional (CW2D) 16
2.11.3 PREWet Model 16
CHAPTER THREE 20
MATERIALS AND METHODS 20
3.1 Study area 20
3.2 River water quality 21
3.2.1 Re-aeration and de-aeration rate 22
3.2.2 Critical distance, time to critical distance and critical DO 23
3.2.3. Maximum allowable BOD (L_{aw}) of a river 24
3.3 Subsurface Vertical flow constructed wetland 25
3.3.1 Substrate analysis 25
3.3.3. Sizing of a pilot constructed wetland 26
3.3.4 Wetland water budget 27
3.4 SODIS system 27
3.5 Experimental set up of the pilot VF CW 28
3.6 Evaluating performance of the system 29
3.7 Sample analysis 30
3.7.1 Laboratory Analysis 30
3.7.2 Microbial parameters 31
3.8 Data analysis 32
3.9 Predicting various scenarios Using PREWet model 32
3.9.1 Model inputs and output 32
3.9.2 Sensitivity analysis 32
3.9.3 Model Simulation 33
3.9.4 Prediction using PREWet model 34
CHAPTER FOUR 35
RESULTS AND DISCUSSION 35
4.1 River water quality 35

4.1.1 Physico-chemical parameters 35
4.1.2 Nutrient concentrations in Njoro River stretch 36
4.1.3 De-aeration, re-aeration and self purification ability 38
4.1.4 Critical distance (X_c), critical DO and allowable BOD 39
4.2 Performance of the existing Egerton University constructed wetland 41
4.3 Pilot system set up 43
4.3.1 Substrate Media 43
4.3.2 Water budget 44
4.4 Performance of the VF Constructed Wetland system 45
4.4.1 Physico - chemical parameters results from the pilot CW system 45
4.4.2 Nutrients concentration in the Pilot CW 46
4.4.3 Performance of the constructed wetland on faecal coliforms removal 48
4.4.4 Performance of the system at different Hydraulic retention times 48
4.5 PREWet Model Performance 54
4.5.1 Model calibration 54
4.5.2 Sensitivity analysis 54
4.5.3 Model simulation 56
4.5.4 Model prediction 60
CHAPTER FIVE 63
CONCLUSION AND RECOMMENDATIONS 63
5.1 Conclusion 63
5.2 Recommendations 64
REFERENCES 65
APPENDICES 71

LIST OF FIGURES

Figure 1: Map showing the study area 20
Figure 2: The pilot VF CW system set up 29
Figure 3: Schematic flow diagram of the system 29
Figure 4: Mean nutrients concentration for the four sampling sites along Njoro River. 38
Figure 5: NO_3–N Concentration for influent and effluent of the system at different retention times 49
Figure 6: NO_2–N concentration for influent and effluent of the system at different retention times 50
Figure 7: NH_4-N Concentration for influent and effluent of the system at different retention times 51
Figure 8: TP concentration for influent and effluent of the system at different retention times 52
Figure 9: SRP Concentration for influent and effluent of the system at different retention times 52
Figure 10: Effect of changing retention time on PREWet model output (plug flow option)... 55
Figure 11: The r^2 of observed and simulated concentrations of TP 57
Figure 12: The r^2 of observed and simulated concentrations of Coliform 57
Figure 13: The r^2 of observed and simulated concentrations of BOD_5 58
Figure 14: The r^2 of observed and simulated concentrations of TSS 58
Figure 15: The observed against the simulated concentrations of BOD_5 59
Figure 16: The observed against the simulated concentrations of TP 59
Figure 17: The observed against the simulated concentrations of coliform 60
Figure 18: The observed against the simulated concentrations of TSS 60

LIST OF TABLES

Table 2-1: Characteristics of WSPs 7
Table 4-1: Mean physico-chemical parameters for Njoro River 35
Table 4-2: Mean nutrient concentration values per site 37
Table 4-3: Self purification parameters 39
Table 4-4: Critical values and allowable BOD_5 40
Table 4-5: Mean influent and effluent concentration ± Standard deviation of Physico – chemical parameters 41
Table 4-6: Mean nutrient parameters of influent and effluent from existing Egerton University CW 42
Table 4 -7: Substrate media characteristics for VF CW systems 44
Table 4-8: Means ± standard deviation of physico – chemical and microbial parameters for influent and effluent from vegetated and the control system 45
Table 4-9: Mean concentrations ± Standard deviation of the nutrients for influent and effluent from vegetated and control system 47
Table 4-10: Removal efficiencies (RE %) in Vegetated and control system for different retention times 49
Table 4-11: Model decay rate factors at different retention times for the modelled parameters 54
Table 4-12: Effects of changing hydraulic retention time on PREWet model output (fully mixed option) 55
Table 4-13: Effects of changing water depth on PREWet model performance (Fully mixed option) 56
Table 4-14: Effects of changing water depth on PREWet model Performance (Plug flow condition) 56
Table 4-15: Model prediction on Nakuru data using plug flow condition 61
Table 4-16: Model prediction on Nakuru data using fully mixed condition 61

LIST OF PLATES

Plate 1: Hydraulic conductivity set up 26
Plate 2: System set up 28
Plate 3: Expanded clay 44

ACRONYMS

BOD	Biological Oxygen Demand
CWs	Constructed Wetlands
DAAD	Deutscher Akademischer Austausch Dienst (German Academic Exchange Service)
DO	Dissolved Oxygen
EC	Electrical Conductivity
GIS	Geographical Information System
GPS	Global Positioning System
HLR	Hydraulic Loading Rate
HRT	Hydraulic Retention Time
NH_4-N	Ammonium nitrogen
NO_2-N	Nitrites
NO_3-N	Nitrates
NEMA	National Environment Management Authority
PREWet	Pollutant Removal Estimates for Wetlands
ROSA	Resource Oriented Sanitation
SODIS	Solar disinfection
SPSS	Statistical Package for Social Sciences
SRP	Soluble Reactive Phosphorous
SS	Suspended Solids
TDS	Total Dissolved Solids
TP	Total Phosphorous
TSS	Total Suspended Solids
UV	Ultra Violet
VF	Vertical Flow
WHO	World Health Organisation
WSPs	Wastewater Stabilization Ponds
WWTP	Wastewater Treatment Plants

DEFINITION OF TERMS

BOD_5	Biological Oxygen Demand of a 5 day period which is a measure of amount of oxygen required for oxidation of biodegradable organic matter present in water sample by aerobic biochemical action.
Wetlands	Land where the water surface is near the ground for long enough periods each year to maintain saturated conditions and support vegetation adapted to the area.
Constructed wetlands	These are wastewater treatment systems composed of one or more cells in a built and partially controlled environment.
Denitrification	Biotic conversion of nitrate to nitrogen gas.
Influent	Water flowing into a wastewater treatment system
Effluent	Water flowing out of the wastewater treatment system
First order kinetics	This means that the rate of removal of particular pollutant is directly proportional to the remaining concentration at any point within the wetland cell.
Hydraulic residence time	It is average time that water remains in the wetland and expressed as mean volume divided by mean outflow rate.
Nitrification	Biotic conversion of ammonium nitrogen to nitrite and nitrate nitrogen.
Subsurface flow wetland	This is an excavated basin filled with a porous media usually gravel in which the water level is maintained below the top surface.
Surface flow	The type of flow in which the water is exposed to the atmosphere.
Eutrophication	This is excessive growth of aquatic plants due to increase in concentration of plant nutrients in a water body.
Vertical flow CWs	These are vegetated systems in which flow of water is from top to bottom through the substrate mostly through gravitational force.
Wastewater	Water that has been utilised and discharged from domestic dwellings, institutions, commercial establishments and from industries.
Wastewater polishing	It is the process of increasing the environmental compatibility and quality of effluent from the preceding treatments
Wetland hydraulics	Refers to movement of water through constructed wetlands including volumes, forces, velocities, flow rates and flow patterns.

CHAPTER ONE
INTRODUCTION

1.1 Background

Fresh water resources all over the world are threatened not only by over exploitation and poor management but also by ecological degradation (Crites et al., 2006). Water is a precious resource essential for life though getting it in potable form is a big challenge (Kunst and Mudrack, 1996). There is need to protect and conserve this resource and develop alternative re-use options which can help in sustaining its supplies. In spite of their importance, water courses throughout the world are frequently used for disposal of treated, untreated, or partially treated wastewater. Wastewater is generally a major source of ground and surface water pollution, especially in developing countries since their treatment is never comprehensive enough to remove all pollutants from it. Given the rapid spread of water pollution and growing concern about water availability, the links between quality and quantity of water supplies have become more apparent (Smol, 2002). According to Rao and Datta (1989), water pollution is a major problem in many water courses especially those that experience reduced flows. The reduction in flows leads to reduced dilution capacity thus minimising natural self purification ability of a river. One such case is observed in Njoro River which frequently experiences reduced flows.

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Previously, Njoro River which drains into Lake Nakuru flowed throughout the year with high water levels and supported a balanced aquatic plant and animal life. According to Kundu (2007), human population within Njoro River catchment area grew by approximately 400% between 1979 and 1999. In addition, there has been a notable increase in both industrial and agricultural activities within the catchment. Further to this, heavy deforestation of large forest areas in the river's catchment, opening it up for farming and settlement, has lowered water storage that used to seep or flow into the river from the forests thereby reducing its flow. As a result, this river is receiving huge amount of organic and inorganic pollutants from industries, domestic wastewater, agricultural and livestock activities. The organic pollutants should normally be biologically decomposed in the presence of dissolved oxygen in river water as part of its self purification activities. When the quantity of such pollutants is too high, it creates oxygen sag which leads to anaerobic conditions, an initial sign of primary pollution. According to UNEP (2005), conventional treatment plants have primary aim of protecting the fresh water from degradation. Wastewater Stabilisation Ponds (WSPs) have been reported by

USEPA (1988) to have limited nutrient removal. This is even a much greater challenge when WSPs are overloaded due to increased wastewater generation. For instance, the wastewater generation from Municipal Council of Nakuru (MCN) and Egerton University has increased over the years due to increased population in the two institutions. However, the WSPs that were used to treat the wastewater when the generation was low are still the same being used, making them overloaded. This has led to the release of partially treated effluent from these WSPs to the receiving water bodies. The released partially treated effluent contains high nutrient load that ends up into water bodies further robbing them of the naturally dissolved oxygen. This leads to secondary pollution and is considerably more damaging to the oxygen level than the primary pollution. The effluent from Egerton University wastewater treatment plant drains into Njoro River while the effluent from MCN (Kaloleni) WSPs directly flows into Lake Nakuru. This wastewater has high nutrient load which should be reduced to prevent eutrophication of the receiving river and lake. According to Reed et al. (1995) the principal nutrients causing secondary pollution are nitrogen, phosphorous and potassium.

Secondary pollution is highly influenced by how much of these nutrients are added to water. Constructed Wetlands CWs are used to take up such nutrients among other materials before the wastewater is released to surface waters. CWs have been reported in developed countries to reduce nutrients in wastewater when used in secondary or tertiary treatment levels. It was important therefore to test the viability of this option in Kenya. Without good quality water, sustainable development and environmentally sound management of water resources cannot be achieved (Manahan, 2000). This study explored a possible sustainable alternative for polishing effluent from wastewater treatment plants by using both the constructed wetlands and solar disinfection for the removal of micro-organisms.

1.2 Statement of the Problem

In most developing countries, effluent from Wastewater Stabilisation Ponds (WSPs) is mostly discharged into water bodies. Sometimes the discharged effluent acts as reliable recharge source to such water bodies in general. But the wastewater treatment through these ponds hardly removes all the pollutants from them. The quality of the effluent therefore rarely meets the standards required for its discharge into the receiving water bodies. Efficiency of WSPs in removal of nutrients and pathogens is influenced by their hydraulic performance which in turn is affected by the loading of such facilities with the wastewater. While the human population in Egerton and MCN has increased in the recent past, the capacity of the existing

wastewater treatment plants has not been increased. As a result, the hydraulic performance and efficiency of these treatment plants have deteriorated over time and thus greatly reducing the pollutants removal.

1.3 Objectives

The main objective of this study was to evaluate the hydraulic performance of a Vertical Flow Constructed Wetland (VF CW) system in polishing of effluent from wastewater stabilisation ponds.

1.3.1 Specific Objectives

The specific objectives were to:

i. Determine the key parameters necessary for sizing and constructing an efficient model of Constructed Wetland (CW) system for tertiary treatment of effluent from Kaloleni and Egerton University WSPs.
ii. Evaluate the performance of constructed wetland system under various hydraulic loading rates in effluent polishing.
iii. Simulate polishing of effluent from CWs using PREWet model.

1.3.2 Research Questions

i. Which parameters are most important in influencing the efficiency of a CW system, in adequately polishing the effluent from WSPs?
ii. How do different hydraulic loading rates in a CW system affect the wastewater treatment efficiency?
iii. How well does a PREWet model simulate the polishing of effluent from the CWs?

1.4 Justification

The quality of freshwater in most water bodies deteriorates due to the release of raw or partially treated wastewater into these water bodies. Though some of these receiving water bodies, especially rivers, do contribute to the removal of pollutants through self-purification, the extent of this treatment process is dependent on the flow regime of such rivers and the vegetation, mainly the periphyton growth in them. The efficiency of this self purification is further influenced by the flow regimes. The main nutrients in wastewater whose removal depends on the growth of algae, biofilms and the microbial activities in the WSPs are nitrogen and phosphorous compounds. The removal processes for these nutrients together

with some micro-organisms are inefficient in most wastewater stabilisation ponds hence, the CWs have been further used to reduce these nutrients in the wastewater. The CWs enhance the removal of nutrients when proper design capacity and hydraulic performance are put into consideration during their construction. The substrate medium used in the constructed wetlands also plays an important role in reduction of the pollutants in the wastewater. Phosphorous reduction in wastewater as it flows within the wetland is affected by the amount of calcium and iron in the substrate media. Significant clay in the substrate used in CWs also plays a major role in adsorption of phosphorous. Clay has a significant amount of iron and calcium but has a tendency of clogging. To reduce clogging of CWs, expanded clay which has these elements can be used successfully in reduction of the phosphorous.

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CHAPTER TWO
LITERATURE REVIEW

2.1 Introduction

Various anthropogenic activities which include industrial, agricultural and domestic activities require water. After use of water, its quality deteriorates through the introduction of various pollutants (Hochstrat *et al.*, 2008) producing wastewater which is discharged into water bodies and the environment. The major consequence of wastewater discharge to water bodies is the degradation of local streams, rivers, lakes, seas and oceans. Water courses receive pollutants from many different sources which constitute of both point sources and non-point sources. Point sources include wastewater disposal and industrial effluents, while non-point sources may include runoffs and leachates from agricultural farms and grazing areas. According to Horan (1991), identification of major sources of pollutants is important as they can be controlled easily at the source of generation. Water pollution can be a serious health hazard causing sickness to people from drinking, washing or swimming in polluted water bodies (Radojevic and Bashkin, 1999; Manahan, 2000). The risk of water pollution can be reduced by proper treatment of wastewater which can be achieved through properly designed and operated wastewater treatment systems. This would ensure that the hydraulics of the systems are taken into consideration. Tertiary treatment of wastewater using wetland and SODIS system can further enhance the removal of nutrients and faecal microbes thereby ensuring that the discharged effluent has low microbial contamination. When constructed wetlands are used, they reduce the nutrient levels in the effluent which then curbs the problem of eutrophication in the receiving water bodies.

2.2 Eutrophication

The increase in concentration of plant nutrients in a water body that results into excessive growth of aquatic plants is known as eutrophication. According to Hammer and Hammer (2004), introduction of nutrients in water is a natural and desirable process that allows for growth of aquatic organisms in rivers and lakes. However, human activities can greatly increase the presence of nutrients, resulting into very high levels of nitrogen and phosphorous being discharged into water bodies. This causes an increase in aquatic plants growth, thereby giving rise to algal blooms especially in standing water bodies. The algal bloom leads to de-oxygenation of water when they die since micro-organisms breaking down the decaying organic matter resulting from this big biomass uses up oxygen in the water thereby causing a reduction of dissolved oxygen levels in water. Eutrophication problem can be solved by

ensuring that wastewaters discharged into water bodies do not contain excessive nutrients level. One way of achieving this is by employing effective methods of wastewater treatment capable of removing these nutrients. Such method may involve a combined use of constructed wetlands with selected plants such as vetiver that are known to be more effective in the uptake of plant nutrients.

2.3 Wastewater treatment

With increasing population and industrial expansion, high production of wastewater from household and industries has been experienced. The increasing human water requirements and demands for cleaner environment make wastewater treatment essential (Vonsperling, 1996) with a potential of reusing such treated wastewater. Although polluted water is unsuitable for drinking, recreation, agriculture and industry, it can be an important resource for use in selected irrigation after adequate treatment. When wastewater is safely disposed it may act as a reliable recharge point to the water bodies.

Conventional wastewater treatment plants usually have three stages of treatment namely; primary, secondary and tertiary levels. In primary treatment stage, physical operations such as screening and sedimentation are used to remove the floating and settleable solids present. In secondary treatment stage, biological and chemical processes are used to remove most of the organic matter. In tertiary treatment an additional combination of unit operations and processes are used for microbial reduction and removal of nutrients such as nitrogen and phosphorous (Hammer and Hammer, 2004).

2.3.1 Wastewater treatment in developing countries

Small communities in developing countries face significant challenges with regard to wastewater management (Sedlack, 1991) due to the high costs involved in setting up such plants. A wide range of centralized wastewater treatment methods are used in developing countries, including stabilization pond systems, septic tanks, anaerobic systems and land application systems (VonSperling, 1996). The most widely used treatment systems are stabilization ponds because conventional advanced tertiary treatment is expensive, requiring large capital investments to build and operate (Reed *et al.*, 1995). On the other hand, the use of CWs is a relatively new technology with very few studies carried out to investigate its efficiency and therefore potential application for wastewater treatment. In order for resource-scarce developing countries to adopt a given wastewater treatment system, the treatment

technologies involved must be cost-effective and easy to adopt, require less energy input, and maintenance costs and be capable of meeting effluent discharge standards. One of the viable options that exists and was tested is the use of wetland treatment system combined with solar disinfection (SODIS) technique for the removal of pathogenic microbes.

2.3.2 Wastewater stabilisation ponds (WSPs)

These are also known as wastewater lagoons or oxidation ponds. They are a system of lagoons arranged in series through which wastewater flow as solid wastes settle down while organic matter are microbially broken down to release plant nutrients which are taken up by aquatic plants. They are mostly employed as secondary treatment systems. They act as polishing ponds for mechanical treatment plants. The pre-treatment of the wastewater to the ponds include flow measurements and screening (Hammer and Hammer, 2004). WSPs are classified according to the biological activities taking place in them as facultative, aerobic and anaerobic ponds. The tertiary pond includes the maturation ponds or polishing ponds. Table 2-1 summarizes the unique features of these treatment ponds.

Table 2-1: Characteristics of WSPs

Parameters	Anaerobic	Facultative	Maturation
Design	2 – 5 m deep with a small surface area.	1.5m deep, shallow but large surface area	About 1m but with a large surface area
Flow	Hydraulic retention time of 1 – 3 days	Hydraulic retention time of 10 – 20 days	Retention time of 10 days
Function	Sedimentation Anaerobic stabilisation of sludge (BOD reduction of 40 – 50 %). Main function is settling	Aerobic degradation of suspended and dissolved matter. Main function is degradation	Sedimentation of suspended solids i.e. bacteria mass and pathogen. Useful also in nutrient removal. Main function is hygienisation.

Source: Hammer and Hammer (2004).

2.4 Effluent polishing and disinfection

Effluent polishing is treatment process used to remove additional suspended solids and BOD from the secondary effluent. This is because sensitive receiving waters promote a need for

effluent polishing. This may include chlorination, wetland and UV or solar disinfection. To polish the pre-treated wastewater, wetlands are effective in reduction of the nutrients.

After secondary treatment of wastewater, microbial contamination still remains. The purpose for disinfecting wastewater effluent is to protect public health by inactivation of pathogenic organisms such as eccentric bacteria, viruses and protozoans. The most popular disinfectants are chlorine (gas and liquid) and ultraviolet (UV) radiation (Hammer and Hammer, 2004). UV radiation in areas with long yearly sunlight hours can be very effective though the initial installation cost is high.

2.4.1 Chlorination method

This is the most widely used method in chemical disinfection. Chlorine contact consists of rapid initial mixing, which creates a high initial kill at the point of contact (Elkarmi *et al.*, 2008). Precise measurement of chlorine residual is extremely important for efficient and economical operation. Monitoring and feed back control prevents low concentrations and inadequate disinfection as well as excessive chlorination, resulting in discharge of an effluent toxic to aquatic life. Also in chlorination more processes like de-chlorination are required before disposal of effluent to receiving water bodies. For better results to be achieved there is need for close monitoring of the entire process. Since chlorine use produces trihalomethane (THM) compounds, its use should be minimised to protect the environment. Use of UV or solar disinfectant to disinfect wetland effluent was explored since this does not produce THMs.

2.4.2 Solar disinfection (SODIS)

This method uses the ultraviolet radiation from the sun to kill microbial pathogens in water. Ultraviolet (UV) light within a narrow frequency (320 – 420nm) contains energy that when absorbed in the DNA of micro organism it changes the DNA structure hence preventing reproduction at the cellular level. UV light is effective in limiting the reproduction of coliform bacteria (Wegelin and Sommer, 1998). A study conducted by Elkarmi *et al.* (2008) showed that 4 – 6 hours exposure is adequate for inactivation of faecal bacteria to be achieved. Turbidity and temperature have an effect on the efficiency of UV radiation in inactivation of the micro-organisms in contaminated water. If the water is too turbid, the micro-organisms will only be exposed to the heat effect which is not enough to kill them. According to Mashauri *et al.* (2000), effluent from CWs is of low turbidity but requires

disinfection to reduce the number of coliform bacteria. One easy method involves the exposure of transparent plastic bottles of water to the sun.

2.5 Wetlands

This is a more natural way used to remove certain nutrients and polishing of effluent from Wastewater Treatment Plants (WWTPs) before it flows into the receiving water bodies. Wetland plants through their canopy, biomass and rhizosphere create an environment that supports and promotes a wide range of physical, chemical and microbial processes that have the effect of purifying the water as it flows through the wetland bed. The processes separately and in combination remove Total Suspended Solids (TSS), trap and degrade organic matter, and lower the Biological Oxygen Demand (BOD). In addition, these processes transform nitrogen forms, provide storage for metals, cycle phosphorous and attenuate organisms of public health significance. The biogeochemical cycling of macro and micro nutrients within the wetland is the framework for the treatment capability of a wetland system (Wetzel, 2004).

2.6 Wetlands plants

According to Greenway and Woodly (2000), plants play important roles in the functioning of wetland treatment system. Apart from their aesthetics values, plants take up nutrients thereby purifying the water in the wetlands. Since aquatic plants have a natural mechanism for pumping air through their root system, the root area provides an oxygen rich environment which supports a range of aerobic bacteria similar to those found in other wastewater treatment processes (Loveridge *et al.*, 1995). Various types of aquatic plants that have been used in wetland systems designed for wastewater treatment that thrives in Kenya includes; canna lily, cattail, reeds and vetiver system.

i. Canna lily *(Zantedeschia aethiopica)*

Canna lily plants do well on rocky media system but do not grow in standing open water. They have succulent stalks and leaves that attract every kind of predator and disease. Once they are established they are very difficult to eradicate from an area. Studies show that they do very little in treatment except by transpiring water. Due to their soft tissues, they increase the BOD and nitrogen in the effluent of the wetland once they die off (Surrency *et al.*, 2003).

ii. Cattail (*Typha Spp)*

The cattail plants have salinity tolerance, surviving an optimum pH ranging from 4–10 and they are drought resistant. Their growth is rapid through the rhizomes and spread laterally. They have relatively shallow root penetration in the gravel. This limits their use in the subsurface flow systems especially when design depth of the wetland bed is not adjustable.

iii. Reeds (*Phagmites communis*)

The reeds are tall annual grasses with extensive perennial rhizomes. They rapidly grow through rhizomes and have deep root penetration in the gravel. The rhizomes penetrate vertically in gravel. They are drought resistant and have low food value for most birds and animals. They survive at optimum pH of 2 – 8 and a salinity tolerance of $45gL^{-1}$.

iv.Vetiver system (*Vetiveria zizanioides)*

Vetiver grass has extraordinary and unique morphological and physiological characteristics, including a deep rooting system with a strong soil binding capacity and stiff stems. It adapts to almost any soil and climate. It is not invasive because of its sterility and lack of rhizomes. It is useful in wastewater treatment, pollution and waste management. It has remarkable characteristics which include a high level of tolerance to elevated and even toxic levels of salinity, acidity, alkalinity, sodicity and a whole range of heavy metals and agro chemicals. It has exceptional ability to absorb and tolerate elevated levels of nutrients (Truong *et al.*, 2008). Vetiver system is useful in soil and water conservation though it uses more water than other common wetland plants. There was need however to test its effectiveness in its performance on the CWs in polishing effluent from the wastewater ponds in Kenya. This is because most of the available literature on its performance is based on pilots in Australia and China.

2.7 Constructed wetlands (CWs)

These are wastewater treatment systems with one or more treatment cells designed and built in a partially controlled environment. They have flow beds that have substrate which helps in supporting plants adapted to areas with a lot of water (Langergraber and Haberl, 2001). The CWs are engineering structures used worldwide to improve water quality (Kadlec *et al.*, 2000; Langergraber and Haberl, 2001). CWs are among the proven efficient technologies for wastewater treatment (Kivaisi, 2002) that are of low cost, easily operated and maintained, and

have a strong potential for application in developing countries as compared to conventional treatment systems (Kadlec and Knight, 1996).

The CWs are of two types, the Vertical Flow and Horizontal Flow wetlands depending on direction of water flow through the substrate. The vertical flow CWs are vegetated systems in which the flow of water is from top to bottom through the substrate and the effluent is collected at the bottom. Vertical flow CWs have been in use in domestic wastewater treatment for several years (Baede–Bederski *et al.*, 2004; Gross *et al.*, 2007; Langergraber, 2008). It has been reported that such vertical flow beds are extremely effective in removing suspended solids, BOD_5 and for nitrification of ammonia (Brix and Arias, 2005). They have been put into use because of their ability to provide high level secondary and tertiary wastewater treatment at a fraction of land use of horizontal Subsurface Flow (SSF) or the Surface Flow (SF) wetlands. Vertical flow systems have much greater oxygen transport capacity and therefore provide much better nitrification (Suthersan, 2001). On the basis of the flow paths of the wastewater through the CWs treatment system, two types of CWs namely the Surface Flow (SF) and Subsurface Flow (SSF) CWs are identified.

2.7.1 Surface Flow (SF)

The Surface Flow (SF) wetlands have water flows at low velocity above and within substrate in shallow channels and basins. The basins normally contain a combination of gravel, clay or peat based soils and crushed rocks, planted with vegetation. They are usually 15 to 30 cm deep (Langergraber, 2007). Since water is exposed to the atmosphere, the SF can act as breeding sites for mosquitoes. In addition there is also the risk of odour and public contact with partially treated wastewater.

2.7.2 Subsurface Flow (SSF)

The Subsurface Flow (SSF) constructed wetlands have water that flows beneath the surface in the root zone and are specifically designed for the treatment or polishing of the wastewater. The substrate used in their construction is mostly gravel, sand and other soils laid at a depth ranging from 30 to 90 cm. This substrate supports the root structure of the emergent vegetation. The wastewater in SSF constructed wetlands has contact with the surface of the media and plant rhizosphere. The subsurface water level inhibits the breeding of mosquitoes and odours from wastewater. It also reduces risk of public contact with the partially treated wastewater. The microbial reaction rates in SSF wetlands are higher than in SF wetlands. As

a result, they release effluent with better quality and surface requirement for treatment system is decreased (Kadlec *et al.*, 2000). Effluent from vertical flow CWs is of better quality compared to effluent from horizontal SSF CWs. This is because horizontal SSF CWs has a problem of clogging and leads to short circuiting between flows (Langegrabber, 2005).

2.8 Design of Subsurface flow constructed wetland

According to Reed *et al.* (1995), well designed CWs can treat wastewater to levels that are suitable for safe reuse or disposal to receiving water bodies. The design of a treatment system is based on physical and chemical characteristics of the inflowing wastewater and the required effluent policy. When treatment systems are designed to function properly, they protect the environment in a socio-culturally and economically sustainable manner (Langergraber, 2007).

The design of constructed wetlands has been described by Reed *et al.* (1995) and Kadlec and Knight (1996). The main difference between Reed *et al.* (1995) approach and that of Kadlec and Knight (1996) is the basis of their choice of reaction rate constants. Reed's Equation uses a volumetric, temperature dependent basis with their calculation being based on the available volume of the wetland and the average water temperature. The Kadlec and Knight (1996) equation assumes an areal basis; thus the rate constant is related only to the wetland surface area, and temperature changes are considered significant only for nitrogen removal. The capacity of wetlands to treat wastewater is limited, both in terms of the quantity of water, and the total quantity of the pollutants. Hydraulic overloading occurs when the water flow exceeds the design capacity. This causes a reduction in water retention time that affects the rate of pollutant removal. Pollutant overloading occurs when the pollutant input exceeds the process removal rates within the wetland (White and Shirk, 1998).

The performance of any CW system is dependent upon the system hydrology as well as loading rate. A hydrologic budget should be prepared to properly design a CW treatment system. Changes in detention time or water volume can significantly affect the treatment performance. Precipitation, Evapotranspiration, Hydraulic Loading Rate (HLR) and water depth can all affect the removal of organics, nutrients and trace elements not only by altering the detention time, but also by either concentrating or diluting the wastewater. Rain causes two opposing effects, including dilution of waters, thereby reducing material concentrations, and increased water velocity, decreasing the water retention time within a wetland. Changes

in retention times within the wetland significantly affect the treatment performance for a CW. The water balance to a wetland can be calculated using equation 2-1 (Reed *et al.*, 1995).

$$\frac{dv}{dt} = Q_i - Q_e + P - ET \qquad (2\text{-}1)$$

Where Q_i= Influent wastewater flow (volume/time)

Q_e= Effluent wastewater flow (volume/time)

P = Precipitation (volume/time)

ET= Evapotranspiration (volume/time)

v = Volume (m^3)

t = Time (days)

The equation does not consider the inflow to and from the groundwater, since the SSF wetlands should be lined. According to Metcalf and Eddy (2003), BOD is still a commonly used parameter in sizing of the constructed wetlands. Both Reed *et al.* (1995) and Kadlec and Knight (1996) admit that BOD_5 removal in SSFCW can be described by using first-order plug flow kinetics. The BOD removal is given by the first order plug flow model (equation 2-2) which can also be used to determine the Hydraulic Retention Time (HRT). However, pilot studies should be carried out before using the plug flow model to design full scale CW in tropical countries. This is because the available coefficients for the model are from the temperate countries.

$$\frac{C_e}{C_i} = \exp(-K_T t) \qquad (2\text{-}2)$$

Where C_e= Effluent pollutant concentration, mg/l

C_i= Influent pollutant concentration, mg/l

t = Hydraulic Residence Time, days

K_T= Temperature–dependent first order reaction rate constant, per day which is calculated as given in equation 2-3.

$$K_T = K_{20}(1.06)^{T-20} \; . \qquad (2\text{-}3)$$

Where K_{20} for SSF is first order reaction rate constant at 20^0C and given as K_{20} = 1.104/day as recommended by Reed *et al.* (1995).

In determining the surface area of the wetland, there is need to determine the hydraulic conductivity and the porosity of the substrate. This helps in establishing the wetland retention time which affects the hydraulics of the system. The hydraulic conductivity helps in determining the head loss within the system. Darcy's equation gives the relationship between flow through the porous media and hydraulic gradient, which assumes the laminar flow through the media (USEPA, 1988).

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2.9 Pollutant removal in CWs

As wastewater flows through the substrate which is composed of soil, sand, rock or artificial medium, purification processes occur through contact with the surface of the media and plant rhizospheres. The processes involved can be physical, biological or chemical. According to Brix and Arias (2005), CWs are efficient in removal of suspended solids, BOD_5 and nitrification of ammonia. Phosphorous removal in wetlands is not very effective and the principal mechanism for its removal is through plant uptake and retention in the soil (USEPA, 1988). Phosphorous removal in CWs is an exception when proper soils are selected as the medium for the system since limited removal is experienced when the substrate used is rock and sand based systems (USEPA, 1988).

A significant clay content and presence of iron and aluminium can enhance phosphorous removal. However, most of the substrates used in CWs (pea gravel, crushed stones, sand, gravels) do not contain high concentration of calcium, aluminium and iron content and as a result, phosphorous removal is generally low (Vyamazal, 2002). The concentration of oxygen however affects this phosphorous adsorption process in the clay particles through the processes of redox potential. Though redox potential does not affect phosphorus' oxidation state, redox potential is indirectly important because of its effect on iron solubility (through reduction of ferric oxides). The levels of oxygen affect the processes of redox potential. Low oxygen leads to decreased redox potential and pH values. The increase in pH decreases phosphorous binding capacity of iron and aluminium and hydroxide ions replace phosphates. Calcite formation at higher pH values increases the phosphorous binding capacity of calcium. In order to improve the phosphorous retention in CWs, substrates with higher phosphorous adsorption capacities, larger particle surface area and suitable hydraulic conductivity should

be used (Vyamazal *et al.*, 1998). In this study, the potential of expanded clay as a substrate in CWs was explored for reduction of phosphorous.

Expanded clay is an artificially fabricated material composed of clay baked at high temperature to create light pellets of clay with high specific surface area and high porosity (Ulimaier *et al.*, 2009). Normal clay can not be used effectively as a substrate in constructed wetlands due to its tendency of clogging.

2.10 Self purification ability of a river

Most flowing water courses are capable of absorbing a certain amount of pollutants without suffering adverse effects through growth of aquatic plants and biofilms (Ifabiyi, 2008). The ability of a water body to absorb and degrade organic pollutants is a measure of its self purification capacity. When this capacity is exceeded, rivers start to exhibit signs of pollution, as a result oxygen sag curves result (James, 1993). The oxygen sag profile along the length of a watercourse is mirrored by the composition of the aquatic biotic community within the watercourse and pollutants discharged into it. This is as a result of re-aeration and de-aeration of the water as it flows through its course and the uptake of oxygen by the micro-organisms as they breakdown the organic pollutants introduced into the rivers. The oxygen sag curve gives oxygen concentration at any point along a body of water below the point of waste discharge. The main points of interest to engineers on this curve are; oxygen deficit at a given point downstream of the discharge, the value of critical oxygen deficit and the time at which the critical oxygen deficit is reached (Horan, 1991). This information is useful to engineers in designing wastewater treatment plants that can achieve the set standards and the objectives of water quality management. The effect of the effluent on the receiving water vary greatly from environment to environment, thus the concept of a universally or nationally applicable standard is unsound.

2.11 Wastewater treatment models

Integrated water quality management is only possible when the impact of wastewater discharged to the aquatic water ecosystem can be predicted quantitatively by means of integrated wastewater models (Jogernsen and Bendoricchio, 2001). Some of these models include the Continuous Flow Stirred and Plug Flow design models. In Continuous Flow Stirred Model, a completely mixed up condition is maintained in the system. This is achieved by stirring continuously to ensure that no concentration gradient is set and that the contents

are homogeneous at every point. In addition, each particle has a different retention time, with some leaving before the mean hydraulic retention time and others remaining longer. This model greatly differs with the conditions of a wetland. Plug Flow Design Model considers effluent as passing along the length of the treatment without experiencing any longitudinal mixing (Horan, 1991). Since there is no intermixing, every element in the system experiences treatment for equal time period. In this model, the response is based on the Hydraulic Residence Time (HRT) and on the temperature in the system. Though the wetland is not an ideal plug flow reactor, the response tends to be closer to plug flow than to the completely mixed alternative.

2.11.1 Wetland Water Quality Model (WWQM)

This is a simulation model that evaluates nitrogen, phosphorous and sediments retention from a constructed wetland system. It is divided into four sub-models which include; hydrological, nitrogen, phosphorous and sediment. It calculates nutrients, hydrological parameters and sediments based on available data, assumptions and knowledge of the wetland system. The model is used for non point source pollution control and considers the watershed area (Trepel *et al.*, 2000).

2.11.2 Constructed Wetland 2 - Dimensional (CW2D)

It was developed to model the biochemical transformation and degradation process in SSF CWs (Langergraber, 2005). This is a multi-component reactive transport model CW2D (Constructed Wetlands 2D) that describes the biochemical degradation and transformation processes for organic matter, nitrogen and phosphorus. The mathematical structure of CW2D is based on the International Water Association (IWA) Activated Sludge Models (Henze *et al.*, 2000). The model is Data intensive.

2.11.3 PREWet Model

This is based on the plug flow design model and is an analytical, screening-level model which assumes the steady state conditions and one dimensional longitudinally varying concentration. The main objective of the model is to estimate removal efficiency (RE) for a specific pollutant given a limited amount of basic information about the wetland. The model uses simplified quantitative methods that minimise time and effort for the end user (Dortch and Gerald, 2000). The model quantitatively expresses the amount of pollution removed in terms of removal efficiency (RE) given in percentage and gives a measure of pollutant

removal achieved by the wetland. Given the basic wetland characteristics and the pollutant of concern, PREWet estimates the amount of pollutant treatment (Dortch and Gerald, 1995).

$$RE = 100 \times \frac{wl - Qc}{wl} \tag{2-4}$$

Where wl = Total loading of pollutant entering the wetland (i.e. $\sum Q_i C_i$)

Q_i = Water flow rate entering the wetland at point i

C_i = Pollutant concentration of flow entering at point i

Q = Total water flow rate exiting the wetland

C = Pollutant concentration of flow exiting the wetland

The equation is applicable to both point and non-point source loadings since the total load entering the wetland can be computed. The primary assumption made with this model to achieve simplicity is that the wetland is at steady state i.e. flow and concentrations are constant with time. With this assumption the analysis is most valid for determining long term average values of RE (Dortch, 1996). Mean annual input conditions (for example, flows, depth) are consistent with this assumption. Two conditions are assumed for spatial gradients in concentration either, fully mixed (that is, no gradients) or gradients along the main flow axis (longitudinal gradients, but well mixed laterally and vertically). The mass balance equation that fits spatial assumption is stated as;

$$\frac{d(V_C)}{dt} = wl - Q_C - KV_C \tag{2-5}$$

Where V = volume of wetland

C = pollutant concentration in the wetland and flowing out of the wetland for the fully mixed assumption

t = time

K = bulk loss or removal rate of the pollutant due to physical, chemical and biological processes

A one dimensional mass transport equation (neglecting dispersion) is used in derivation of spatial assumption and steady state conditions and is stated as;

$$RE = \left(1 - e^{-K\tau}\right) \times 100 \tag{2-6}$$

Where τ = hydraulic residence time, $\frac{V}{Q}$

The true detention time of water parcels can be less than V/Q due to dominant flow paths that result in dead zones and short-circuiting. The location where the pollutant is introduced in the wetland (a point source load) affects the detention time altering it from the theoretical detention time calculated. In the model, derivation for the calculation of hydraulic residence time is estimated from equation given by Thackston *et al.* (1987);

$$\tau = 0.84\frac{V}{Q} = \left(1 - e^{-0.59\frac{L}{W}}\right)$$

For steady state assumption the equation used is stated as;

$$Qc = \frac{wl}{1 + K\tau} \tag{2-7}$$

For the fully mixed option used in the model the governing equation used in calculation of RE is stated as;

$$RE = \left(\frac{K\tau}{1 + K\tau}\right) \times 100 \tag{2-8}$$

Well mixed conditions tend to be associated with wetlands having small residence times and small length to width ratio while plug flow conditions are expected with large length to width ratio and large residence times.

PREWet model addresses BOD, TSS, Coliform bacteria, TN, TP and other contaminants (organic chemicals and trace metals). The total phosphorus removal rate coefficient is either specified by the user or computed from first principles (given the appropriate input information). The assumptions used in developing the algorithm for computing the Total Phosphorus (TP) removal rate coefficient in PREWet is based on the assumption that the growth and death of aquatic plants have no net effect on the long-term removal of TP. The justification for this is that plants take up phosphorous when they grow, but recycle it when they die. So for steady-state conditions, there is no net effect. TP is assumed to partition only between water and solids, and the processes of suspended solids deposition, sediment re-suspension, diffusion dissolved phosphorus across the sediment-water interface, and sediment

burial affect its fate. A steady-state mass balance for the water column and sediment bed is used to derive the relationship of TP removal rate for these processes.

The RE depends on the wetland retention time and the removal rate, K (day^{-1}), for the constituent. Other hydraulic variables needed include flow velocity, hydraulic depth, and the water surface area. The wetland hydraulic depth, H (m), is defined as;

$$H = \frac{V}{A} \tag{2-9}$$

Where A = water surface area (m^2)

V = volume (m^3).

Thus, with an estimate of two of the three variables (V, A, and H), the third variable can be computed. The mean velocity of the flow, U, is either input by the user or estimated from;

$$U = \frac{Q}{WH} \tag{2-10}$$

The removal rate (K) also depends on the pollutant of concern and wetland characteristics (Dortch and Gerald, 2000). Removal rate, K is dependent on a number of processes such as microbial metabolism, adsorption, volatilization, denitrification and settling. These processes are affected by ambient conditions such as water temperature and pH. The model computes wetland outflow concentrations for each constituent when inflow concentrations are entered. The PREWet model is robust, has less memory space requirement and is not data intensive.

CHAPTER THREE

MATERIALS AND METHODS

3.1 Study area

The study was conducted at the Municipal Council of Nakuru (MCN) and Egerton University Njoro Campus; where the focus was on the Kaloleni and Egerton University wastewater stabilization ponds (Figure 1). Egerton University is located at 0° 22'19.48"South, 35°55'46.8"East at an average elevation of 2265m. The MCN is located at 0°19'41.08" South 36°05'32" East, and at an average elevation of 1755m. The University has a population of about 15,000 persons (Egerton Water and Sewerage Department, 2009), while that of MCN is about 800,000 persons (CBS, 2009).

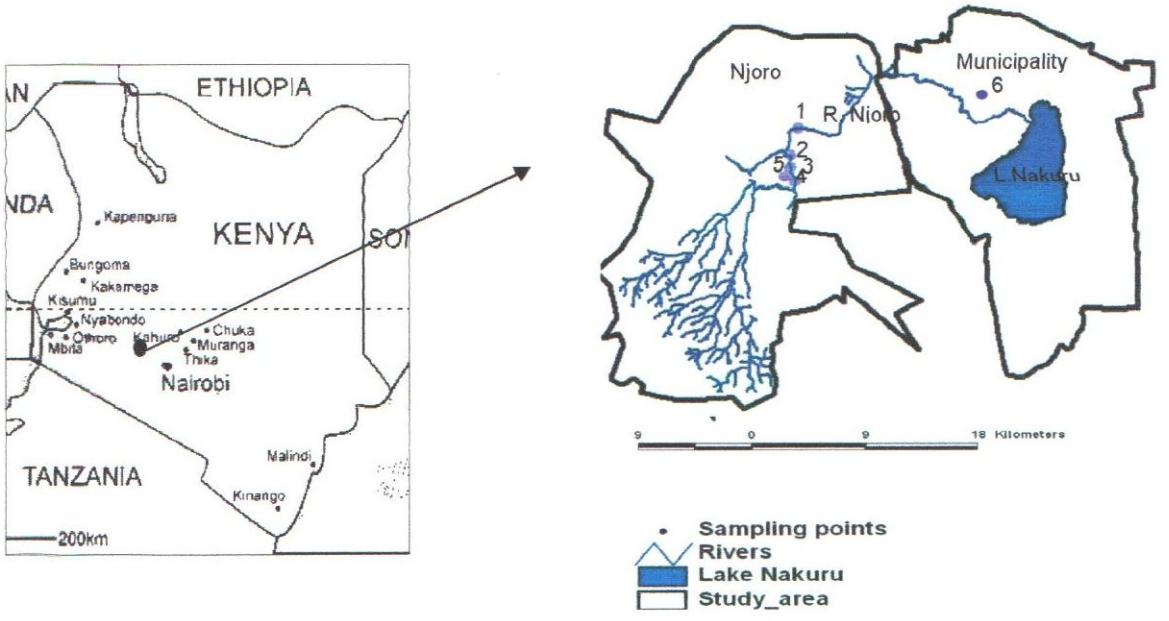


Figure 1: Map showing the study area
The sampling points; 1= Njoro Bridge, 2= Bora Milk, 3= Before Njoro Canning Factory, 4= 100m after discharge point, 5= Effluent from Egerton WSP's and 6 effluent from Kaloleni WSPs

Study sites

The study sites were a stretch of Njoro River, Egerton University and Nakuru Kaloleni Wastewater Stabilisation Ponds (WSPs). The Kaloleni treatment plant in Nakuru receives wastewater from non–industrial sources, mainly domestic wastewater from residential areas in the municipality. The plant has a capacity of 16,200m^3/day. It comprises two anaerobic, two facultative and six maturation ponds with rock and grass filters. The effluent flows directly into Lake Nakuru and acts as a major source of water and nutrients that promote the heavy growth of algae in the lake. The effluent from Kaloleni WSPs also supports the wild

animals with drinking water especially during the dry season. Egerton University has WSPs comprising two anaerobic ponds, with the old and new ponds receiving $37m^3$/day and $680m^3$/day of wastewater respectively. In addition there are two facultative ponds and two maturation ponds. The university generates about 717 m^3/day of wastewater which is eventually released into Njoro River after treatment (Egerton Water and Sewerage Department, 2009). Njoro River receives effluent from wastewater treatment plants, which in the middle reaches include effluent from Egerton University and Njoro Canning wastewater treatment plants. Njoro River is a second order stream, 60km long with little Shuru as its main tributary (Mathooko, 2001; Shivoga, 2001). The river originates from Eastern Mau at an elevation of about 2700m. and drains into Lake Nakuru at an elevation of 1700 m a.s.l. at the floor of Rift valley. It acts as an important source of water to the rural riparian inhabitants for domestic uses and watering of their livestock.

Njoro River catchment area is approximately 200 km^2 and experiences pronounced temporal seasonal patterns in rainfall. This accounts for high annual and seasonal variability of Njoro River flow. The catchment receives a mean annual rainfall of 600-1200 mm and temperatures ranging between 9°C - 24°C. The stream flow ranges from high flows during transient storm events in the wet season to the cessation of flow in the lower reaches during dry spells.

3.2 River water quality

To determine the water quality of Njoro River, four sampling points were identified along a 5km stretch of the river between Njoro town and Egerton University Bridge. The coordinates of the sampling points were taken using a Global Positioning System (GPS) in Universal Traverse Mercator (UTM) format. Point 1 was located at 9962884m S, 827685m E, point 2 at 9960970m S, 827146m E, point 3 at 9960076m S, 827156m E and point 4 at 9959184m S, 827498m E. The sampling points were selected at sites which had minimal or no flow obstruction in the upstream. Sampling was carried out for five months (October, 2009 to February, 2010). Samples were taken weekly for the first two months and then after every two weeks for the remaining period. 500 ml plastic bottles were used for sample collection. The bottles were pre treated by washing them in dilute hydrochloric acid and then rinsed thoroughly with distilled water. They were then autoclaved since samples were to be analysed for microbial contamination.

During sampling, the bottles were rinsed three times with the relevant samples of water and then filled with the samples. Parameters such as Dissolved Oxygen (DO), pH, temperature, salinity, Total Dissolved Solids (TDS) and Electrical Conductivity (EC) were measured in-situ using their respective meters. The discharge of the river was determined using the current meter method.

The data collected at the sampling sites along the river was used to establish the river's self purification ability after introduction of pollutants from Egerton University wastewater discharge point. The parameters required to establish self purification ability of the river included; critical distance, time to critical distance and the critical dissolved oxygen of the river. These parameters together with de-aeration and re-aeration rate constants were calculated.

3.2.1 Re-aeration and de-aeration rate

The de-aeration rate, K_1 of Njoro River was calculated using Rao and Datta, (1989) equation given as;

$$K_1 = \frac{1}{t}\log\frac{l_a}{l_b} \qquad (3\text{-}1)$$

Where t = time of travel in days between stations a and b,

l_a= BOD measured at station a

l_b= BOD measured at station b.

Re-aeration rate K_2 was calculated from modified Streeter and Phelps equation;

$$K_2(20) = 3.9\frac{V}{H^{\frac{3}{2}}} \qquad (3\text{-}2)$$

Where V = mean stream velocity (m/s)

H = mean depth (m)

3.9 = a factor to correct for changes in surface roughness of the bed and

$$K_2 = K_2(20)\theta k_2^{(T-20)} \qquad (3\text{-}3)$$

Where $K_2(20)$ = re-aeration rate at 20^0C

θk_2 = temperature coefficient usually 1.0241

K_2 = re-aeration rate at a given temperature

T = Temperature of the water

Effects on concentration of Dissolved Oxygen (DO) within the stream by rate of addition of Biological Oxygen Demand (BOD) through local runoff and product of bottom sludge deposits were calculated as given in Appendix A using equations A-1 to A-4.

3.2.2 Critical distance, time to critical distance and critical DO

To determine the critical distance of Njoro River, Streeter and Phelps (1989) equation was used. The equation is given as;

$$x_c = \frac{v}{K_1(f-1)} \ln\left\{ f\left[1-(f-1)\frac{Da}{La}\right]\right\} \quad (3\text{-}4)$$

Where f = the self purification ratio

x_c = Critical distance

Da = Dissolved oxygen at station a

La = BOD at station a

The self purification ratio was calculated from de-aeration and re-aeration rates;

$$f = \frac{K_2}{K_1} \quad (3\text{-}5)$$

Where K_1 and K_2 are defined in equations 3-1 and 3-3

From re-aeration and de-aeration rate calculated above, the time to critical distance was calculated using James (1993) equation. The equation is given as;

$$t_c = \frac{K_1 \log\left(\frac{K_2}{K_1}\left(1-DO_0(K_2-K_1)\right)\right)}{K_2 \quad K_1 BOD_0} \quad (3\text{-}6)$$

Where t_c = time it takes DO to get to critical point

BOD_0 = initial BOD

DO_0 = Initial DO deficit of the mixture of river water and wastewater

DO_0 had been calculated using equation below;

$$DO_0 = DO_s - \frac{Q_w DO_w + Q_r DO_r}{Q_w + Q_r} \quad (3\text{-}7)$$

Where DO_s = Saturated value of DO in water at temperature of the river

Q_w = Volumetric flow rate of wastewater (m^3/s)

Q_r = Volumetric flow rate of the river just upstream of point of discharge (m^3/s)

DO_w = DO in the wastewater

DO_r = DO in the river just upstream

DO_S was also calculated as;

$$DO_s = 14.652 - 0.41022T + 0.00799107T^{-2} - 0.000077774T^{-3} \quad (3\text{-}8)$$

Where T = Temperature of river water in °C

After determination of time to critical distance, the Dissolved Oxygen at that critical point (D_c) was calculated. This was calculated as given by Rao and Datta (1989).

$$D_C = \frac{La\exp(-K_1 t_c)}{f} \quad (3\text{-}9)$$

Using self purification ability of the river and critical distance, the degree of treatment required for wastewater from Egerton University was then calculated.

3.2.3. Maximum allowable BOD (L_{aw}) of a river

Maximum allowable BOD (Law) of effluent from wastewater treatment plant was calculated from the stream flow and wastewater discharge data. Complete mixing condition was assumed at the point of discharge. L_{aw} was then calculated. The coefficients F_3, F_4 and DO at any point along the river (D) were calculated using equations A-7, A-8 and A-9 given in Appendix A.

$$L_{aw} = \frac{D - F_3(1 - F_2) - DaF_2}{F_4(F_1 - F_2)} + \frac{Lc}{(K_1 + K_3)} \quad (3\text{-}10)$$

Where D= DO at any point along the river in a given time

When the maximum allowable BOD (L_{aw}), of the river was determined, maximum allowable BOD of effluent from Egerton University wastewater treatment plant, L_w was then calculated;

$$L_{aw}(Q+q) = L_S Q + L_w q \quad (3\text{-}11)$$

Where Q = rate of flow in the stream

q = rate of discharge of the waste

L_s = BOD of the stream, upstream to the point of discharge.

L_w = BOD of the effluent from wastewater treatment plant

L_{aw}= Allowable BOD of the stream

3.3 Subsurface Vertical flow constructed wetland

A Subsurface vertical flow CW was sized and constructed using sand, expanded clay and gravels as the substrates. Expanded clay, which is artificially fabricated clay fired at high temperature ranging between 950°C and 1200°C (Arioz *et al.*, 2008) was prepared. Clay and sawdust was mixed at a ratio of 40:1 (40kg of clay mixed with 1kg of sawdust). Moulds of 20 x 15 x 10cm were prepared, cured for 21 days and then fired in the kiln. The resulting bricks were then crushed and sieved to get grain size of diameter ranging between 2 – 9 mm.

3.3.1 Substrate analysis

Hydraulic conductivity, Ks, porosity, n and volume of voids, V_V were determined. The porosity value was necessary in determining the cross-sectional area available for flow in a wetland.

i. Hydraulic conductivity (Ks)

The constant head conductivity test was carried on the substrate media used. A K-610 permeameter and K670 constant level tank was used in determination of Ks. This consisted of a standard compaction mould, collar, mounting base with a porous base and brass pipe fitting. It also had a head with air escape valve and fitting. Water was allowed to rise slowly in the specimen and was seen slowly rising in pipette. After the water in pipette attained steady state, the water height and level of outlet were taken. The set up used for determination of Ks was as shown in Plate 1. The discharge, Q was then determined for a known period of time. The Ks was calculated using equation;

$$Ks = \frac{QL}{AH} \qquad (3\text{-}12)$$

Where Ks= Hydraulic conductivity, cm/sec

Q= Rate of discharge, cm^3/sec

L= Length of specimen, cm

H= Pressure head, cm of water

A= Area of specimen, cm^2

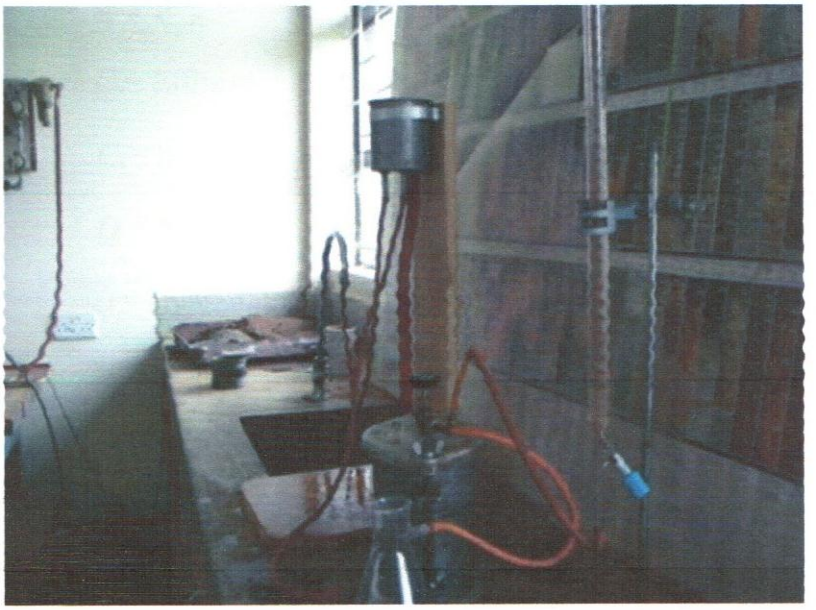


Plate 1: Hydraulic conductivity set up

ii. Volume of voids, Vv and Porosity, n

Water saturation method was used in determining the porosity of the substrate. The total volume of the substrate was obtained by measuring the total volume of the sample using a 500ml beaker, while the volume of voids was evaluated directly by measuring the volume of water required to completely saturate the samples. The porosity obtained was confirmed using water evaporation method where the volume of voids was obtained as the difference in the weight of saturated sample and the weight of the dried sample. The porosity was calculated by dividing the water added with the volume of the sample (500 ml) using equation;

$$n = \frac{Vv}{V} \tag{3-13}$$

Where n = Porosity

Vv = Volume of voids

V = Volume of sample used

3.3.3. Sizing of a pilot constructed wetland

A first order plug flow model was used in sizing the constructed wetland. The BOD_5 of wastewater from last maturation pond and the calculated allowable BOD_5 discharge to the river were used as influent and effluent concentration respectively in plug flow model to calculate HRT. The depth of water in wetland was then calculated using equation;

$$y = Q\frac{(\ln C_i - \ln C_e)}{A_s K_T n} \quad (3\text{-}14)$$

Where C_e= Effluent pollutant concentration, mg/l

C_i = Influent pollutant concentration, mg/l

Q = Average flow rate

t = Hydraulic Residence Time, days

y = depth of water in wetland

A_s = surface area of the wetland

n = porosity

K_T =Temperature dependent 1st order reaction rate

HLR of the wetland was calculated using the equation given in Reed *et al.* (1995).

$$HLR = \frac{100Q}{A_s} \quad (3\text{-}15)$$

3.3.4 Wetland water budget

Wetland water budget was established using meteorological data collected from Egerton weather station. Some of the important meteorological data included evaporation rate, precipitation and temperature. The Evaporation data collected using class A pan was converted to potential evapotranspiration, ET_0.

$$ET_0 = kET \quad (3\text{-}16)$$

Where k = crop factor in this case grass

According to Reed *et al.* (1995), the k for subsurface CW is 0.8. This study adopted this value in the conversion of evaporation data. The evaporation and precipitation data collected was also converted into volume per time using area of the wetland. The flow into and out of the wetland was recorded through out the study period. Plastic tanks were used thus infiltration and ground water effect to the water budget was negligible.

3.4 SODIS system

Polyethylene (PET) bottles were filled with water and placed on a dark sheet of metal exposed to the sun. This helped in increasing the amount of heat within the system thus

improving on disinfection. Water was allowed to remain in the bottles for approximately 5 hours as recommended by Elkarmi *et al.* (2008). Samples were then collected and analysed for microbial contamination. SODIS was run for one month only during the study since the effluent from CWs was noted to have low microbial contamination.

3.5 Experimental set up of the pilot VF CW

The experiment was set up in Egerton University and the pilot wetland used effluent from University's wastewater stabilisation ponds. Sub surface Vertical flow CW in this study was composed of granular material with different grain sizes arranged in layers of varying heights. The substrate used had three layers; sand, expanded clay and gravel. The cover layer was sand which facilitated water distribution over the entire wetland; the main layer was expanded clay and gravel was used in the drainage layer. Vetiver grass was planted in the wetland. The set up had three replicates and one control. The control had no vetiver grass planted on it (Plate 2). This was useful in evaluating performance of vetiver grass in nutrients reduction. To ensure uniform wastewater distribution to the wetland units and out of the wetland, the flow was regulated. This was achieved by use of ½" gate valves which were fitted on the flow distribution pipe and on the outlet of the wetland (Figure 2).



Plate 2: System set up

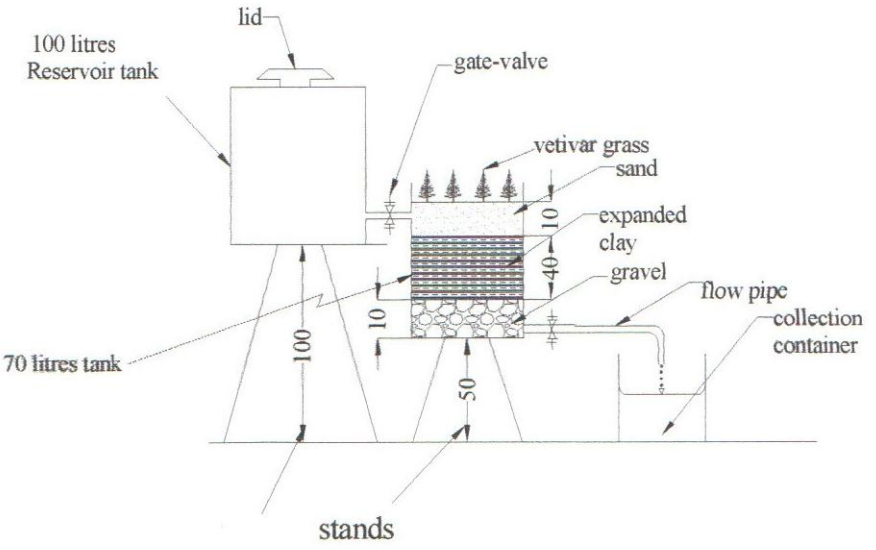


Figure 2: The pilot VF CW system set up

3.6 Evaluating performance of the system

The wetland system was evaluated based on its treatment performance. Flows through the system were regulated to control the loading rate. Different flow rates were used and thus different treatments of the retention time were applied. The change of flow rate helped in establishing optimal retention time required for the treatment of all the pollutants other than the BOD_5 which was used in sizing of the system. Samples were collected at sections 1, 2 and 3 of the system as shown in Figure 3. Samples from sections 1 and 2 were analysed for physico-chemical, nutrients and biological parameters while those from section 3 were analysed for microbial contamination. The results were compared with the standard set by Water Resources Management Authority (WRMA) and National Environment Management Authority (NEMA).

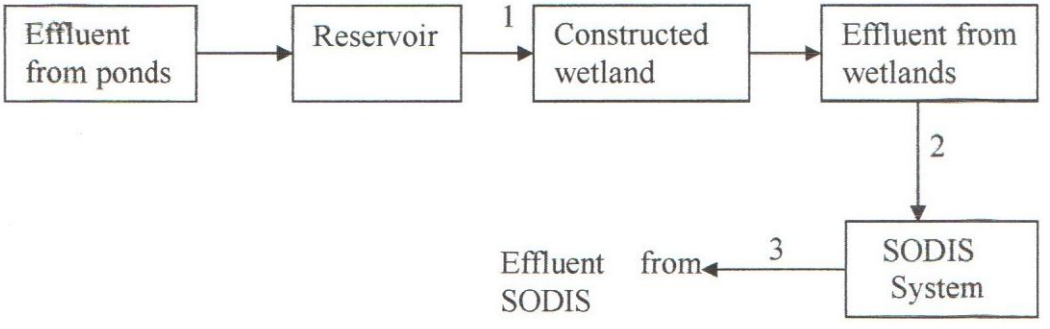


Figure 3: Schematic flow diagram of the system

3.7 Sample analysis

Water samples collected during the study were analysed for physico-chemical, nutrients and microbial contamination. In-situ parameters analysed during samples collection were; Dissolved Oxygen (DO) mg/l, pH, temperature °C, salinity g/l, Total Dissolved Solids (TDS) mg/l, and Electrical Conductivity (EC) µs/cm which were measured using their respective calibrated meters. The DO and pH were measured using HQ40d dual input, multi parameter digital meter with pHC101 and LDO101 probes. This meter was also used for temperature measurement. TDS, EC and salinity were measured using VWR EC300 meter.

3.7.1 Laboratory Analysis

The following nutrients were analysed; nitrate nitrogen (NO_3-N) mg/l, nitrite nitrogen (NO_2-N) mg/l, Total Phosphorous (TP) mg/l, Soluble Reactive Phosphorous (SRP) mg/l, and Ammonium nitrogen (NH_4-N) mg/l. Biological Oxygen Demand, mg/l (BOD_5), Total Suspended Solids, mg/l (TSS), organic content and inorganic contents were also determined. Feacal coliform was analysed using the Most Probable Number (MPN) method within 2 hours after sampling.

In the laboratory, the collected sample was divided in two. The sample to be used for SRP, NO_3-N, NO_2-N, and NH_4-N was filtered using Whatmann GFC filters of 47mm diameter and pore opening of 0.45µm. The filter was first pre-combusted and weighed. A known volume of water sample was filtered and the filter dried. This was weighed and TSS calculated as given below. The filter was then combusted at 500°C for 4 hours and weighed again in order to calculate the organic and inorganic content (APHA, 1995).

$$TSS = \frac{(Wc - Wf) \cdot 10^6}{V} \tag{3-17}$$

Where TSS = total Suspended Solids (mg/l)

Wc = weight of filter and residue (g)

Wf = weight of filter (g)

V = volume of the sample filtered (ml)

Unfiltered sample was used for TP analyses which was first digested and reduced into free Ortho-phosphate form using the Persulphate digestion for 90 minutes at 121°C. After reduction, it was then analysed using ascorbic acid method as recommended in APHA, (1995). SRP was analysed using ascorbic acid method on the filtered sample. NH_4-N was determined using the reaction between sodium salicylate and hypochlorid solution while NO_3-N was determined using the Sodium-salycilate method (APHA, 1995) with standard solutions of the nitrate prepared for the standard calibration curve. NO_2-N was also determined using reaction between sulfanilamid and N-Naphthyl–ethylendiamin-dihydrochlorid which gave an intense pink colour with the presence of nitrite (Appendix B-1). The nutrients concentration in the treated samples was determined through light absorbance from the colour complexes formed after reactions with the reagents at specified wavelengths for the given samples on a LKB 4053 Ultrospec K spectral photometer.

The BOD_5 was determined using the Winkler method (Appendix B-2) to measure the oxygen content in the samples. The DO of diluted sample was measured before and after incubation for a period of 5 days since BOD_5 is commonly used. The measurement of BOD was carried out in "Dark flasks" to prevent the incidence of light thus inhibiting primary production by phytoplankton which would have generated oxygen in the process. The BOD_5 was then calculated;

$$BOD_5 = \frac{D_1 - D_2}{P} \tag{3-18}$$

Where D_1 = DO of diluted sample immediately after preparation (mg/l)

D_2 = DO of diluted sample after 5 days incubation (mg/l)

P = Volumetric fraction of sample used in decimal

3.7.2 Microbial parameters

The presence of faecal coliforms in samples was analysed within 2 hours after sampling using MPN method. This aided in determining the sanitary quality of the water sample. The samples were diluted up to 0.01. Autoclaved multiple tubes that had 5ml of medium A-1 were used and 1ml sample was added and incubated at 44°C for 24 hours. The numbers of positive tubes (with gas production) were then counted and MPN of faecal coliforms per 100ml of sample was determined from a Table given in Appendix B-3.

3.8 Data analysis

Analysis of Variance (ANOVA) was used in data analysis at a confidence level of 95% using Statistical Package of Social Sciences (SPSS) version 17 as a tool for analysis. This was carried out to check whether there was a significant difference on the means of treatment efficiency at different Hydraulic loading rates. The significant means in ANOVA were separated using Least Square Difference (LSD) as the post Hoc test. Before the ANOVA was carried out, the Levene homogeneity test was carried out on the data. Spearman rank correlation was used to investigate the relationship between salinity, DO, BOD, pH, temperature and Electrical Conductivity (EC) and the nutrients in the effluent. The Spearman rank correlation was also used to investigate relationship between the simulated and observed data from the PREWet model.

3.9 Predicting various scenarios Using PREWet model

The PREWet model was calibrated, validated, tested for sensitivity and used to predict the performance of the system. Half of the data set collected from the pilot wetland system run in Egerton University was used in calibrating the model and the remaining half of the data set was used in validating the model. The model was calibrated to reduce its performance uncertainty. In the calibration process, the model coefficients values which included temperature correction and decay factors were adjusted in order to match closely with the observed. The decay rate factors were established for the different constituents modelled at different detention times.

3.9.1 Model inputs and output

Inputs to the model were system properties, hydraulic retention time, surface area, depth and volume of the wetland and temperature of the area. The constituents to be modelled were also input which included; BOD, TP, TSS, and coliform bacteria. The inflow concentrations of these constituents were entered. The output of the PREWet model included; removal rate per day, removal efficiencies and the out flow concentration of the constituents modelled.

3.9.2 Sensitivity analysis

The PREWet 2.4 model was subjected to sensitivity analysis in order to identify its critical parameters. The model was tested with several runs for different wetland design parameters. These parameters included wetland dimensions i.e. area, volume, depth, width and length. Each parameter was studied by changing its corresponding value while holding the other

parameters constant. The discharge and retention time that corresponded to the estimated parameters were calculated. To study the effects of changing modelling parameters e.g. retention time and discharge, the removal rate constants were fixed. Initially, one of the wetland dimension parameter was changed keeping retention time to 2 days as had earlier been calculated. This necessitated changing the wetland capacity and discharge, Q. Also Q was maintained as the design value and changing one wetland dimension, different retention time was operated according to the new dimensions.

3.9.3 Model Simulation

The PREWet model was used in simulation of pollutant treatment through the constructed wetland. The input data that was used included the wetland dimensions, retention time and inflow characteristics of the constituents from the actual field data collected. The removal rate coefficients that were obtained during calibration of the model were used during the simulations for the different retention times. Simulation was run using the plug flow and fully mixed options. Values of the collected and simulated data for each constituent (TSS, TP, BOD and Coliform) were used to draw a scatter plot. To validate the model, simulated data from PREWet model was compared with observed data from the pilot wetland run. Correlation analysis of the simulated and observed data was undertaken using the statistical package SPSS 17.0.

The results were compared using the coefficient of model efficiency given by Sutcliffe (1989).

$$COE = 1 - \left[\frac{\sum_i^n (q_w - q_s)^2}{\sum_i^n (q_w - q_m)^2} \right] \qquad (3\text{-}19)$$

Where COE = the coefficient of model efficiency,

q_w = Measured parameter value from the wetland

q_s = Simulated parameter value from model

q_m = Mean of measured parameter values

n = Number of measurements taken

Nash–Sutcliffe coefficient value less than 0.6 indicated the need to calibrate and validate the model.

3.9.4 Prediction using PREWet model

The data from the analyses of physico-chemical, nutrients and microbial contamination from the effluent of Nakuru (Kaloleni) WSPs was used in the PREWet model which had been calibrated from data collected from pilot wetland set up in Egerton University. The data was used as the influent characteristics of the constituents modelled, while removal rates and decay factors were maintained constant in the calibrated model. This was undertaken for all the retention times operated during the study maintaining their removal efficiency.

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 River water quality

4.1.1 Physico-chemical parameters

The results for the river water quality for physico-chemical parameters per site are as given in Table 4-1. The mean *DO* and BOD_5 of the sampling points ranged between 7.01 to 9.02 mg/l and 3.47 to 6.26 mg/l respectively which agreed with the results earlier reported by Yillia and Mathooko (2007) in their study of the Njoro River characteristics.

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Table 4-1: Mean physico-chemical parameters for Njoro River

Sampling site	Conductivity (µs/cm)	pH	Temperature (°C)	Salinity (g/l)	TDS (mg/l)	TSS (Mg/l)	Organic content	Inorganic content	DO (mg/l)	BOD_5 (mg/l)
1	379.26	7.90	17.6	0.22	239.21	50.10	32.18	18.13	7.01	6.26
2	190.34	7.60	17.6	0.10	131.84	26.42	6.45	19.98	8.47	3.68
3	203.79	7.66	17.1	0.11	136.63	21.10	6.50	14.60	9.02	3.47
4	203.23	7.42	17.8	0.11	135.28	39.15	8.8	30.37	8.84	5. 10
Mean	**244.16**	**7.65**	**17.5**	**0.14**	**160.60**	**34.19**	**13.48**	**20.77**	**8.34**	**4.63**

1. 100m after discharge point 2. Before canning 3. Bora milk 4. Njoro Bridge

High values of BOD_5 were recorded at the wastewater discharge point from the University while low levels were recorded near the Bora milk site. Wastewater discharged into the river from Egerton University introduces organic matter into the water thus raising the BOD_5 level. Low BOD_5 levels recorded at the site near Bora milk could be attributed to the self purification effect of the river between the University and this point, through microbial activities, water turbulence that aerates the river and the activities of periphyton in taking up adsorption and assimilation of the inorganic and organic compounds from the water. At Njoro Bridge, BOD_5 levels could have increased as a result of in-stream activities carried out at this point. These may include discharge of organic matter to the river from animal droppings as they are being watered, laundry activities by the nearby community that also introduce organic matter into the river. Increase in BOD_5 at Njoro Bridge could also be as a result of introduction of organic matter by wastewater from Kenya Agricultural Research Institute (KARI).

Generally, the reduction in the BOD_5 level along a river is normally attributed to biological and chemical breakdown of organic matter through microbial activities into simple minerals.

Biological growth attached to the river bed or biological flocs also take up some of these organic matters from flowing river (Ifabiyi, 2008). This finding agrees with the results of the organic matter content of the TSS determined between the University wastewater disposal point into the river and the Bora milk site point. Higher organic contents in TSS were recorded at the University wastewater discharge point while as the river flows, the organic content in the water at the lower reach decreased while the inorganic contents formed a higher proportion of TSS. Decrease in organic matter content in the TSS could be attributed to their breakdown through microbial activities. The high levels of the organic content at discharge point indicate that the effluent discharged in the river contains higher concentration of organic materials. Higher concentration of TSS was recorded at the discharge point (50.10 mg/l) while at Njoro Bridge, the mean TSS was 39.15 mg/l showing an increase from 21.10mg/l recorded at Bora milk site. The increase in TSS could be attributed to the in-stream anthropogenic activities carried out at Njoro Bridge such as the watering of animals and manual abstraction of water from the river which disturbed the sediments in the river bed.

Dissolved oxygen (DO) levels near Bora milk point were high due to the turbulent nature of the river between Njoro canning and Bora milk site which enhances the dissolution of oxygen into the water from the atmosphere. Temperature along the river reach did not vary significantly, ranging between 17.1 and 17.8°C (Table 4-1). This could be attributed to the presence of riparian vegetation which is not highly interfered with along this stretch thus discouraging direct sun reaching the river water.

The pH of the river water had minimal variations at the different sampling sites while salinity and TDS were high at discharge point and reduced at the remaining sampling points. The minimal variations of pH along the studied stretch could be as a result of buffering ability of water, and also it can indicate that the discharges from the point sources along this stretch could have been within the neutral range.

4.1.2 Nutrient concentrations in Njoro River stretch

Significantly higher concentration of nutrients, especially TP and SRP were recorded at the University wastewater discharge point into the river as compared to the other sampling points. The concentration of these nutrients reduced downstream up to Bora milk point and then increased slightly at the Njoro Bridge (Figure 4). The values recorded for TP at

University's discharge point to Bora milk site ranged between 1.46 to 0.18 mg/l while at Njoro Bridge the mean concentration was 0.22 mg/l. SRP followed the same trend, reducing from 1.28 to 0.07 mg/l and a slight increase to 0.1 mg/l at Njoro Bridge (Table 4-2). SRP contributed about 40% of TP at Njoro Bridge, Bora milk and just before canning while at the university's discharge point it contributed 82%. The self-purification process of the river which occurs through algal uptake of nutrients and sedimentation could have caused the gradual decrease in phosphate concentration along the stretch studied. The noted increase of phosphate levels at the Njoro Bridge site could be from in-stream activities which led to sediments disturbance. Phosphorous could have been introduced at Njoro Bridge site from detergents used as people wash clothes. The increase could also be as a result of wastewater discharged from KARI to the river.

Table 4-2: Mean nutrient concentration values per site

Sites	TP (mg/l)	SRP (mg/l)	$N0_2$-N (mg/l)	NO_3-N (mg/l)	NH_4-N (mg/l)
1	1.46	1.28	0.37	0.99	3.34
2	0.22	0.08	0.29	0.78	0.13
3	0.18	0.07	0.28	0.80	0.10
4	0.22	0.1	0.18	0.62	0.07

1. 100m after University's discharge point; 2. Before canning; 3. Bora milk and 4. Njoro Bridge

NH_4-N levels decreased significantly from discharge point downstream with values ranging from 3.34 mg/l at the University wastewater discharge point to 0.07 mg/l at the Njoro Bridge (Table 4-2). NO_3-N levels also reduced from the discharge point to Njoro Bridge and values recorded were 0.99 to 0.62 mg/l respectively. NO_2-N values from the discharge point downstream the river to Njoro Bridge were 0.37 and 0.18 mg/l which indicated that there was reduction of this nutrient downstream.

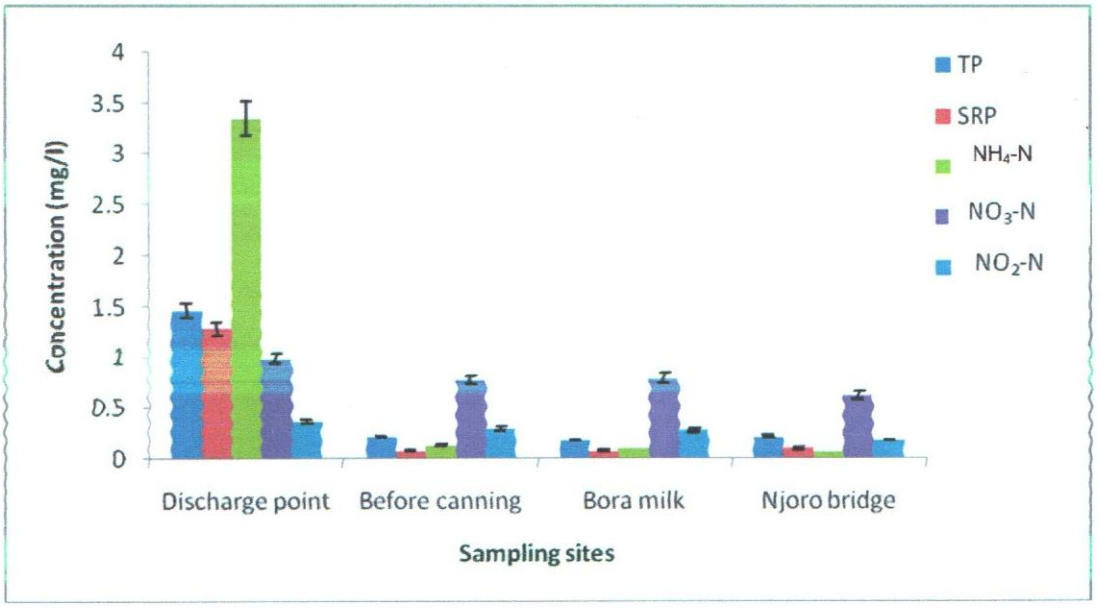


Figure 4: Mean nutrients concentration for the four sampling sites along Njoro River. Vertical lines are ± Standard Error, n=30.

The reduction of nutrients downstream could be due to processes such as sedimentation and volatilization (Akpofure *et al.*, 2006). Sedimentation occurs when the water flow reduces at some sections of the river, thus encouraging the suspended particles to settle at the river bed. Sedimentation encourages the reduction of the particulate phosphorous and as a result the phosphorous nutrient reduces downstream. The volatilization process takes place when the aquatic plants especially the periphytons and microbes take up the nitrates and the ammonium nitrogen. Reduction of pollutants downstream can also be attributed to biological activities in the river causing uptake of nutrients and breakdown of organic matter thereby enhancing the self purification impact of the river. Reduction of ammonia could be due to its uptake by algae and other aquatic plants. Also the nitrification process could have caused a gradual decrease in ammonium concentration along the stretch in question.

4.1.3 De-aeration, re-aeration and self purification ability

De-aeration rate ranged between 0.08 and 0.25/day, with the mean value being 0.14/day. Re-aeration rate ranged between 0.32 and 0.69/day while the mean re-aeration value recorded was 0.47. De-aeration (K_1) and re-aeration rate (K_2) constants were used in calculation of self purification rate (f) of the river (Rao and Datta, 1989). These constants are affected by the depth and flow rate of the river. For the whole sampling period, the mean depth and flow rate of Njoro River was found to be 14.4cm and 0.41m^3/s respectively. The average self

purification rate was calculated to be 3.60. Throughout the sampling period, the re-aeration rate was found to be higher than the de-aeration rate (Table 4-3). This is an indicator that deposition occurs in Njoro River. Self purification ability is affected by sedimentation of suspended solids and oxidation of the soluble materials. Processes of oxidation give rise to oxygenation of river water. De-aeration rate depends on the strength of the wastewater, the degree of dilution afforded by the admixture with the river water, and the velocity of the river. If concentration of oxidisable material is excessive, the river water suffers considerable or complete de-oxygenation and this result into anaerobic decomposition of the organic matter. Sufficient dilution leads to organic matter being oxidised and thus it is destroyed without depriving off the river water oxygen to any appreciable degree. DO in river water plays an important part in the process of purification.

Table 4-3: Self purification parameters

S/N	K_2 (day^{-1})	K_1 (day^{-1})	f
1	0.47	0.25	1.91
2	0.52	0.16	3.25
3	0.55	0.25	2.16
4	0.32	0.12	2.63
5	0.43	0.11	3.75
6	0.35	0.11	3.11
7	0.37	0.80	4.66
8	0.41	0.10	3.94
9	0.56	0.10	5.66
10	0.69	0.14	4.94
Average	**0.47**	**0.14**	**3.60**

Low values of self purification corresponded with the months with higher amount of rainfall recorded. Seasonal variation is a factor in the self purification of rivers. During warmer months, self purification operates more rapidly. This is due to the increased rate of oxidation of organic matter due to greater bacterial activity and removal of ammonia due to plant development. On the other hand if the water is overcharged with wastewater, the nuisance is greater during warmer periods since increased rate of oxidation may lead to de-oxygenation of water.

4.1.4 Critical distance (Xc), critical DO and allowable BOD

The critical dissolved Oxygen (Dc) of the river ranged between 5.2 and 0.6 mg/l with the average value being 2.85 mg/l (Table 4-4). The average saturated DO of the river was

9.51mg/l. Pimpunchat *et al.* (2008) suggested that critical oxygen of a river is usually 30% of the saturated DO. The calculated mean value of critical DO closely agreed with this. Critical DO was slightly lower than the recommended maximum critical oxygen of 3 mg/l (Horan, 1991). This indicated that the stretch of the river at certain sections turns anaerobic which is a sign of the river being polluted. The time to critical distance (tc) and critical DO had a strong negative correlation i.e. the longer it takes to critical point the lower the critical DO. This could be attributed to de-aeration rate also taking long and thus the oxygen levels in the water reducing. During rainy season, critical DO was noted to be generally low. This could be as a result of increased amount of pollutants being discharged or flowing into the river mainly through surface runoff.

Table 4-4: Critical values and allowable BOD_5

S/N	tc (days)	V m/day	xc (km)	Dc (mg/l)	L_{aw} (mg/l)	L_w (mg/l)
1	1.34	2184.44	2.93	2.18	5.71	9.93
2	3.92	2431.49	9.53	0.58	3.02	4.41
3	2.14	2501.97	5.35	3.64	12.87	42.90
4	0.35	1333	0.46	4.41	11.30	19.17
5	2.62	1126.44	2.95	2.73	13.79	20.61
6	1.31	898.37	1.18	5.17	19.84	29.87
7	2.22	1093.14	2.42	3.68	18.34	31.05
8	4.62	949.96	4.39	2.36	15.57	24.29
9	1.32	1560.62	2.05	2.45	13.76	25.50
10	4.62	1759.08	8.12	1.28	11.24	22.54
Average	**2.44**	**1583.95**	**3.94**	**2.85**	**12.54**	**23.03**

Where, tc is time to critical distance, V is the velocity of the stream, xc is the critical distance, Dc is the critical DO, L_{aw} is the permissible allowable BOD of the river and L_w is the allowable BOD values of the effluent from the wastewater treatment plant

Permissible BOD values (l_{aw}) for Njoro River ranged between 3.01 and 19.84 mg/l, with a mean value of 12.54 mg/l. The allowable BOD values (L_w) for the effluent from Egerton wastewater treatment plant was calculated and the range was 4.41 - 42.89 mg/l with a mean of 23.03 mg/l (Table 4-4). The low values of allowable BOD concentration for the effluent from Egerton University wastewater treatment plant, L_w corresponded with the rainy period. This could be as a result of the river receiving more non point pollutants in the surface runoff. For the river water quality to be maintained, the allowable BOD from Egerton University wastewater plant should not exceed 23 mg/l. The L_w varies with seasons which imply that

during the rainy season the university should discharge effluent with lower BOD_5 concentration than during the warmer season. The calculated L_w values are lower than the BOD_5 effluent discharge standards set by National Environmental Management Authority (NEMA) and Water Resources Management Authority (WRMA) of 40 mg/l and 30 mg/l respectively.

4.2 Performance of the existing Egerton University constructed wetland

The influent and effluent from the Egerton CW had a mean BOD_5 of 240.49 ± 112.14 and 236.92 ± 64.10 mg/l respectively. Such high values imply that if such wastewater would be discharged into the river directly, it would adversely affect the river water quality. From the worked out self purification ability of this river, the effluent from Egerton University wastewater treatment plant should have a BOD_5 value of 23.03 mg/l. This shows that the wastewater from the CW still needs some treatment to remove organic matter before it could be discharged into the river. The mean DO level of effluent was 1.51 mg/l a value that was lower than the mean influent DO of 8.10 mg/l (Table 4-5).

Table 4-5: Mean influent and effluent concentration ± Standard deviation of Physico – chemical parameters

Parameter	Mean Influent ±Std dev	Mean Effluent ± Std dev
EC (µs/cm)	0.87 ± 0.19	0.87 ±0.08
TDS (mg/l)	534.86 ±164.08	569.17±53.10
pH	8.75 ± 0.44	8.16 ± 0.47
Salinity (g/l)	0.45 ± 0.05	0.43 ± 0.05
Temp (C)	20.22 ± 2.13	19.87 ± 0.81
TSS (mg/l)	93.82 ± 26.58	61.21 ± 35.24
Organic matter content (mg/l)	75.09 ± 23.53	51.58 ± 26.04
Inorganic matter content (mg/l)	19.09 ± 31.79	9.38 ±13.35
DO (mg/l)	8.10 ± 2.10	1.51 ± 0.43
BOD_5 (mg/l)	240.49 ± 112.14	236.92 ± 64.10
Coliform (MPN/100ml)	1853.18 ± 1392.48	244.42 ± 104.85

This implied that in the wetland bed there was break down of organic matter which took up oxygen. The wetland could have had no algae to release oxygen in the water through photosynthesis while the large water plants released oxygen directly into the atmosphere. As a result of reduced DO concentrations the performance of the micro-organisms on breaking of the organic matter was inhibited. This led to high concentrations of BOD_5 in the effluent. Due to low DO concentrations, the system had a lower effluent concentration of NO_3-N and NO_2-N since denitrification could have been taking place within the system (Table 4-6). There was a remarkable reduction of coliforms from the influent (Table 4-5). The effluent from the wetland met the NEMA discharge standards of 400 mpn/100ml.

The mean influent and effluent concentration of NH_4-N was 6.80 ±4.44 and 4.9 ± 3.52 mg/l. NH_4-N reduction was low and the effluent did not meet the NEMA discharge standards of 1mg/l (Table 4-6). This could be attributed to reduced DO levels within the system thus reduced nitrification processes taking place and as a result higher concentration of ammonium in the effluent.

Table 4-6: Mean nutrient parameters of influent and effluent from existing Egerton University CW

Parameter	Mean influent ± std dev	Mean effluent ± std dev
TP (mg/l)	6.86 ± 0.98	5.85 ±1.05
SRP (mg/l)	6.00 ±1.06	5.28 ±1.11
NH_4-N (mg/l)	6.80 ± 4.44	4.92 ± 3.52
NO_3-N (mg/l)	0.33 ± 0.30	0.31 ±0.23
NO_2-N (mg/l)	0.87 ±0.79	0.40 ± 0.34

From Table 4-6, TP and Soluble Reactive Phosphorous (SRP) concentration of the influent and effluent of Egerton constructed wetland had no significant difference. The TP and SRP mean concentration of the influent was 6.86 ± 0.98 mg/l and 6.00 ± 1.06 mg/l respectively. The mean concentration of the TP and SRP in the effluent was 5.85 ± 1.05 and 5.28 ± 1.11 mg/l. This indicated that there was no or minimal uptake of the nutrients from the wastewater as it passed through the wetland though we have the growth of the wetland plants. The minimal reduction of the TP and SRP could be as a result of wetland plant die off, thus releasing the nutrients back to the water. The TP and SRP concentration of the effluent did not meet the NEMA discharge standards of 2 and 1 mg/l respectively. Minimal reduction of

the nutrients from the existing Egerton University CW was noted (Table 4-6). This could have been due to hydraulic overload (HLR, HRT and flow rate) of the system.

There is need to improve the performance of this wetland especially through the control of its hydraulics (HRT, HLR and flow rate). The design capacity of the wetland system should not be exceeded. This can be achieved by increasing the volume and surface area of the wetland and improve the design of the system. The pilot study conducted on hydraulic performance of a VF CW with expanded clay as the main substrate layer showed clearly that the hydraulics of such a wetland when well worked out drastically improves the quality of the effluents. The results of this pilot system are described below.

4.3 Pilot system set up

The mean BOD_5 concentration of influent and effluent used in plug flow model was found to be 240.49 mg/l and 23mg/l respectively. Hydraulic retention time calculated using plug flow model was found to be 2 days for Egerton university wastewater polishing. The K_T value used in the calculation was 1.12/day, and mean temperature of the wastewater was found to be 20.22° C. Hydraulic Loading Rate (HLR) applied were 1.4, 2.9, 4.3, 8.7 and 17.4 cm/day which corresponded with retention time of 6, 3, 2, 1 and 0.5 days. The calculated HLR was found to be within the two ranges for sub surface flow constructed wetland values given by Kadlec and Knight (1996) and Wood (1995) of 8-30 cm/day and 0.2–3 cm/day respectively. The calculated depth of water in the wetland had a range of 45- 56 cm. This value was within the recommended depth of water in sub surface flow constructed wetland by Langegrabber (2005) and Reed *et al.* (1995), of 30 – 60 cm. This depth of water facilitated flow within the wetland system by providing sufficient head.

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4.3.1 Substrate Media

Usually the substrate media used in CWs do not contain sufficient concentrations of Ca^{2+}, Fe^{3+}, or Al^{3+} to actively adsorb phosphorous (Brix *et al.*, 2001; Vymazal, 2004). The effectiveness of expanded clay as the main substrate layer for the reduction of nutrients depend to some extent on these elements which attach to some of the nutrients especially phosphorus and makes it unavailable to plants. Expanded clay (Plate 3) had grain size ranging between 2–9 mm and gives a porosity and hydraulic conductivity of 0.564 and 0.7568 m/day respectively (Table 4-7). The sand used as the top layer had a hydraulic

conductivity of 0.6682 m/day while that of the gravel used as the drainage layer was 0.82m/day. Hydraulic conductivity varied with the porosity of the substrate media. At low porosity, hydraulic conductivity was low and vice versa (Table 4-7). Porosity and hydraulic conductivity was useful in calculation of the HRT.

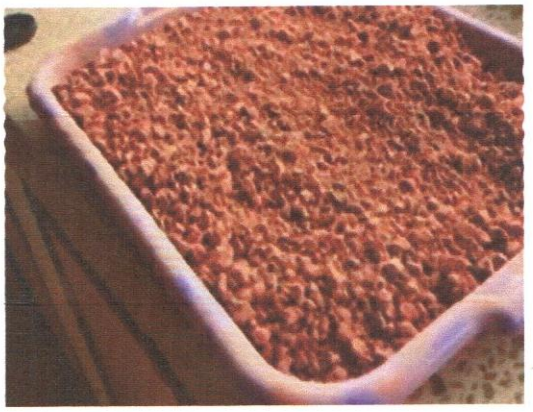


Plate 3: Expanded clay

Table 4 -7: Substrate media characteristics for VF CW systems

Media type	Grain size (mm)	Porosity,n	Hydraulic conductivity, Ks (m/day)
Sand	1 - 3	0.453	0.6682
Expanded clay	2 - 9	0.564	0.7568
Gravel	8 - 16	0.63	0.82

4.3.2 Water budget

From the water budget, the results indicated that hydraulic and pollutant overload was not experienced throughout the study period since water volume within the wetland did not have a significant change. The change in water volume with time in the CW calculated was at an average of 0.0113 m^3/day with an average ET of 0.025 m/day (Appendix C, Table C-3). The low value of ET implies that the concentration of the pollutants was not highly influenced by evapotranspiration. The Hydraulic and pollutant overload negatively affects the performance of the CWs. Hydraulic overloading occurs when the water flow exceeds the design capacity, causing a reduction in water hydraulic retention time that affects the rate of pollutant removal. Pollutant overloading occurs when the pollutant input exceeds the process removal rates within the wetland (White and Shirk, 1998). Evapotranspiration water losses decrease the water volume in the system thus concentrating the remaining pollutants even though the wastewater treatment may be effective on a mass removal basis.

4.4 Performance of the VF Constructed Wetland system

Effective nutrient removal mechanisms including root zone oxidation, adsorption and plant uptake require adequate interaction of wastewater with the treatment media. Subsurface vertical flow constructed wetlands have higher substrate contact with the wastewater being treated, more equal root distribution and water-root contact. As a result there is a remarkable decrease of pollutant concentration in the effluent from these systems (Haberl *et al.*, 1995; Cooper, 1999). The data shows that effluent wastewater loading was significantly lower than influent loading in all treatments ($p \leq 0.001$).

4.4.1 Physico - chemical parameters results from the pilot CW system

Salinity, pH, temperature, TSS, BOD_5, EC, Organic content and inorganic content in the effluent from the vegetated and control system had no significant difference (One way ANOVA, $F_{(2, 103)}$, $P > 0.05$). Mean pH within the two systems studied, the vegetated and control system was 7.18 and 7.18 respectively, while salinity was 0.3 g/l in both systems. According to Kadlec and Knight (1996), wetland waters usually have a pH range of 6-8. From the data collected, pH of the effluent was within this range. There was no significant difference in temperatures of the influent and effluent to the vegetated and control system (Table 4-8). Temperature exerts a strong influence on the rate of chemical and biological processes in wetlands, including organic matter decomposition, nitrification and denitrification processes.

Table 4-8: Means ± standard deviation of physico – chemical and microbial parameters for influent and effluent from vegetated and the control system

Sample		EC(µs/cm)	TDS (mg/l)	pH	Salinity (g/l)	Temp °C	TSS (mg/l)	Organic (mg/l)	Inorganic (mg/l)	DO (mg/l)	BOD (mg/l)	Coli (MPN m
1	Mean	0.68	480.21	7.9	0.37	19.5	66.86	60.14	6.71	5.69	158.39	
	±	0.077	39.13	0.36	0.04	1.2	25.76	21.21	5.88	2.46	99.28	
2	Mean	0.53	404.46	7.18	0.30	19.6	15.97	12.65	3.31	7.29	9.75	
	±	0.07	41.35	0.31	0.04	1.6	8.77	6.39	4.25	1.35	9.05	
3	Mean	0.58	402.56	7.18	0.32	19.0	15.72	10.36	5.36	5.78	10.55	
	±	0.053	35.24	0.14	0.043	1.3	10.13	6.99	3.78	2.32	7.53	

1. Influent 2. Effluent from the vegetated system (mean from replicates) 3. Effluent from the control system

There was a gradual decrease in EC, TDS, pH, TSS and BOD in the vegetated and control system between the influent and the effluents. The TDS decreased uniformly in the vegetated wetland and the control system. The EC generally reduced in vegetated and control systems with higher reductions observed in vegetated system (Table 4-8). This observation agreed

with results reported in Uganda by Kyambadde *et al.* (2004). The decrease in conductivity could be explained by uptake of micro and macro elements and ions by plants and bacteria, and their removal through adsorption to plant roots, litter and settleable suspended particles (Bitton, 1994; IWA, 2000; DeBusk and DeBusk, 2001). The results revealed significant improvement of wastewater's physico-chemical quality owing to the effects of the wetland system. This could be attributed to the diverse chemical and biological activities that take place in various parts of the system.

Dissolved Oxygen in the influent and effluent from the control system had no significant difference ($P>0.05$) but effluent from vegetated system was significantly different from the two ($P<0.05$). Even though the wastewater entering the CWs had lower DO concentration, the effluent from the two systems had higher concentration of DO levels. The highest DO concentration in the effluent was from the vegetated system, where effluent from vegetated and control system had DO concentration of 7.29 mg/l and 5.78 mg/l respectively (Table 4-8). The higher DO levels in the effluent could be attributed to atmospheric aeration and oxygen release during algal photosynthesis. Okurut *et al.* (1999) reported similar trends between unplanted and vegetated systems. The oxygen source for the subsurface components (soil, gravel, rock, and other media, in trenches or beds) is the oxygen transmitted by the vegetation to the root zone. The subsurface flow system is designed to maintain flow below the surface of the bed, thus there is very little direct atmospheric re-aeration. The TSS in the effluent from the two systems was lower than the influent concentration. With low wetland water velocities and appropriate composition of influent solids, suspended solids settles from the water column within the wetland. The physical processes responsible for removing suspended solids include sedimentation, filtration, adsorption onto biofilm and flocculation/precipitation. Wetland plants increase the area of substrate available for development of the biofilm. The rough surface of the plant stems also traps fine materials.

4.4.2 Nutrients concentration in the Pilot CW

A significant difference was observed between the influent and effluent in all the nutrients studied ($p<0.05$) (Table 4-9).

Table 4-9: Mean concentrations ± Standard deviation of the nutrients for influent and effluent from vegetated and control system

Samples	TP (mg/l)	SRP (mg/l)	NH_4-N (mg/l)	NO_3-N (mg/l)	NO_2-N (mg/l)
1	4.71 ± 0.46	3.63±0.49	8.88±4.67	0.13±0.05	0.16±0.15
2	0.95±0.63	0.62±0.58	0.36±0.44	6.67±7.90	0.13±0.20
3	1.16±0.58	0.85±0.64	1.10±1.23	1.52±9.56	0.48±0.51

Key: 1. Influent 2. Effluent from vegetated wetland (mean from replicates) 3. Effluent from control

Reduction in the concentration of SRP, TP and NH_4-N between the influent and the effluent was observed in both the vegetated and control system. The vegetated system had higher reduction of TP and SRP than the control system. This suggests that vetiver grass was useful in reduction of these nutrients through their uptake and perhaps the trapping of organic forms of TP or sedimentation within the plant beds. Most of the phosphorus was in the soluble form shown by the fact that SRP constituted about 80% of TP. A decrease in NO_2-N from the vegetated wetland was noted while in the control system, the NO_2-N increased from the concentration of influent. This could be an indication that nitrification was taking place in the wetland system. Nitrification is known to be favoured in systems with pH and temperature ranges of 6.5–8.6 and 5–30°C, respectively (Schmidt and Belser, 1982; Kadlec and Knight, 1996; Cloete and Muyima, 1997; Grundtz *et al.*, 1998; Im *et al.*, 2001), the condition observed to occur in this system (Table 4-8).

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The reduction of the nitrogen compounds in the vegetated CW was evident in the results collected. NH_4-N was found to decrease more in the vegetated system than in the control system. According to Truong *et al.* (2008), vetiver grass is efficient in the reduction of nitrogen compound. The characteristics of the media type used in this study i.e. expanded clay could have contained higher amounts of Ca, Al and Fe oxides, which were inferred to be a factor causing such high removal of phosphorous by adsorption. Therefore expanded clay can be used effectively as a media, alone or in combination with other materials in CWs. Better performance noted from the pilot wetland run could be as a result of change in substrate media, controlled wetland hydraulics and the wetland plant used in the pilot wetland run.

4.4.3 Performance of the constructed wetland on faecal coliforms removal

Efficiency in the removal of coliform bacteria by the vegetated constructed wetland was shown through a significant difference ($P< 0.05$) between the effluents from the vegetated and control systems (Table 4-8). The effluent from vegetated and control system had feacal coliform concentration of 109 ± 188 and 300 ± 738 MPN/100ml respectively. Vegetated wetlands are more effective in pathogen removal, since they allow a variety of micro-organisms to grow which may be predatory to bacterial pathogens (Reed *et al.*, 1995; Kadlec and Knight, 1996). The processes that may contribute to the removal of coliform pathogens in wetlands include pH changes, natural die-off, sedimentation, filtration, ultra-violet light ionization, temperature effects, and predation by other organisms (Kadlec and knight, 1996). The concentration of the pathogens in the effluent from the CWs and control system was found to meet the NEMA discharge standards of 400 MPN/100ml, thus SODIS system was run for 1 month only. The feacal coliform concentration of the effluent from the SODIS system was 20 ± 20 MPN/100ml. This indicated that SODIS system can be useful in reduction of the bacteria where higher concentration of feacal coliform still lingers in the effluent. However, our system seems to be adequate in the removal of pathogens without involving the SODIS system. Perhaps the open exposure to the solar radiation already contributed to the death of these pathogens.

4.4.4 Performance of the system at different Hydraulic retention times

The reduction of contaminants that is reported in the wetland systems rely on physical, chemical and biological processes. The efficiency of these processes is affected by the residence time of water as it passes through the wetland systems. Treatment performance in constructed wetlands is a function of retention time, among other factors. The actual retention time in constructed wetlands is a function of the porosity of the substrate used. The longer the water remains in the wetland, the greater the chance of sedimentation, absorption, biotic processing and retention of the nutrients. The correct HRT can be achieved by proper sizing of the wetland or by restricting the size of wetland outlet. There was a remarkable decrease in pollutant concentration of TP, SRP and NH_4-N in all flow rates used (Table 4-10).

Table 4-10: Removal efficiencies (RE %) in Vegetated and control system for different retention times

parameter	6 days		3 days		2 days		1 day		0.5 days	
	vegetated	Control	vegetated	Control	Vegetated	Control	Vegetated	Control	Vegetated	Control
TP	91.59	82.98	89.38	81.91	84.27	75.90	75.79	68.24	47.16	40.10
SRP	92.63	89.70	91.73	88.23	86.14	84.85	78.42	64.62	52.79	31.66
NH4-N	95.37	79.51	95.41	90.69	98.61	94.72	99.25	98.79	97.81	95.30
NO_3- N	- 97.62	- 98.07	- 98.23	- 99.29	- 96.70	- 99.27	- 10	- 96.37	- 96.43	- 87.09
NO_2- N	12.85	- 29.61	- 21.40	- 69.37	47.68	- 89.36	- 53.90	88.46	- 45.26	9.39

The flow rates used include 0.012, 0.024, 0.036, 0.072 and 0.144 m^3/day which corresponded with retention time of 6, 3, 2, 1 and 0.5 days respectively. NO_3-N increased in both vegetated and control system (Figure 5).

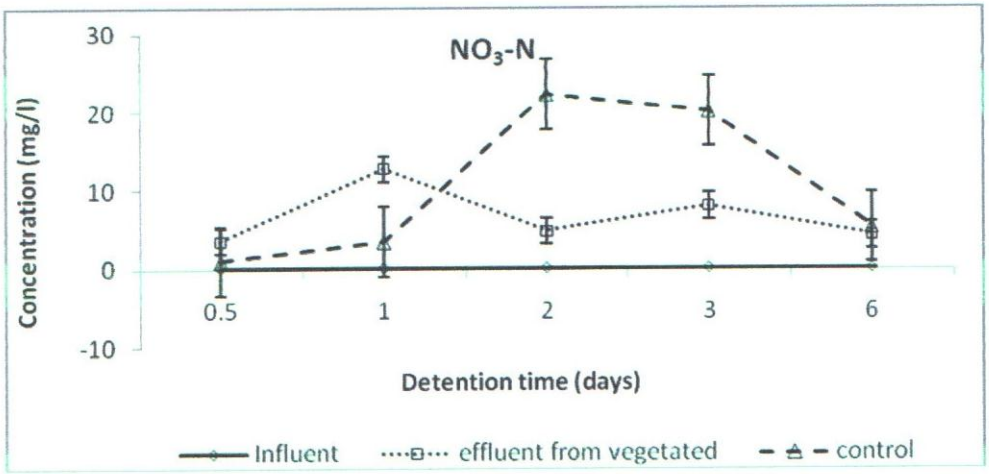


Figure 5: NO_3–N Concentration for influent and effluent from the system at different retention times

Vertical bars = ± Standard errors

The highest percentage increase in vegetated system was at flow rate of 0.024 m^3/day (retention time = 3 days) at a value of 98.23% and lowest percentage increase at flow rate of 0.072 m^3/day (retention time = 1 day) at a value of 10% (Table 4-10). The percentage increase at different flow rates was directly proportional to the dissolved oxygen available. In the control system, the highest percentage increase of NO_3-N was at 0.024 m^3/day flow rate at value of 99.29% and lowest percentage increase at 0.144 m^3/day flow rate (retention time = 0.5 day), with the value of 87.09% (Table 4-10). Effluent from the vegetated CW showed a slight decrease in NO_2-N at retention time of 6, 3, 2 and 1days, while an increase in NO_2-N was observed at retention time of 0.5days (Figure 6).

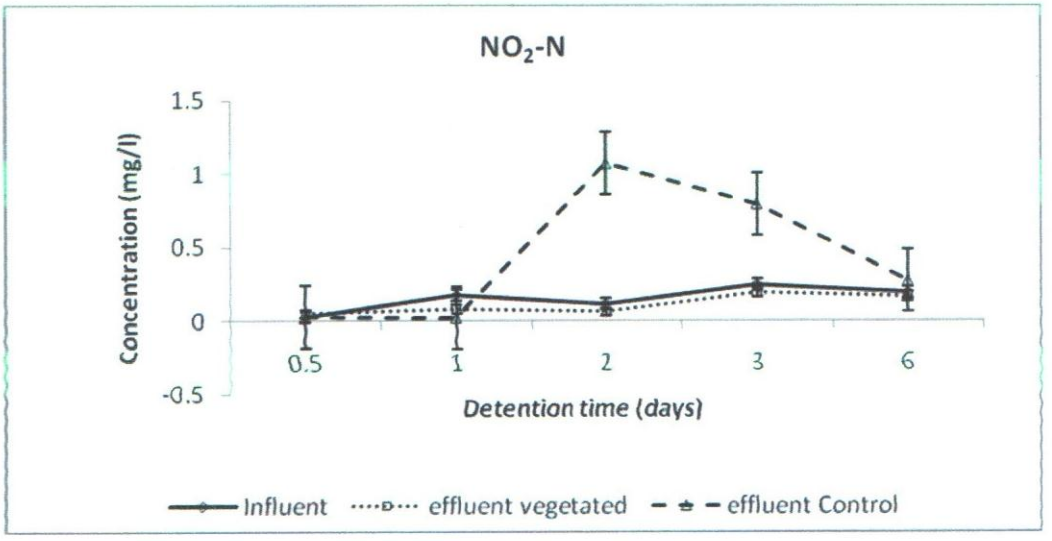


Figure 6: NO_2–N concentration for influent and effluent from the system at different retention times

Vertical bars = ± Standard deviation

The decrease was at 12.85, 21.40 and 47.68% respectively while the percentage increase for NO_2-N was 53.90% and 45.26% respectively. In the control system, NO_2-N increased at retention time of 6, 3 and 2 days with percentage increase of 29.61%, 69.37% and 89.36%. There was a decrease in NO_2-N concentration at retention time of 1 and 0.5 days in the control system. The percentage decrease was 88.46% and 9.39% respectively (Table 4-10). NO_2-N and NO_3-N concentrations increase in relation to the influent concentrations indicates that NH_4-N was probably converted through nitrification to NO_2-N and NO_3-N after passing through the substrate layer. In this study, the rise in NO_2–N and NO_3– N loading in the effluent wastewater compared to the influent wastewater was a manifestation of active system nitrification. The increase of NO_2-N and NO_3-N in wetland effluent was also reported by Kyambadde *et al.* (2005) for a study conducted in Uganda. In wetlands, nitrification occurs in aerobic regions of the water column, soil-water interface, and root zone (Atitaya *et al.*, 2008). The oxygen required for the nitrification process is supplied by diffusion from the atmosphere and release by macrophyte roots.

A reduction of NH_4-N was noted in effluent from both vegetated and control system although NH_4-N removal at retention time of 1 and 0.5 days in vegetated and control system was almost similar (Figure 7).

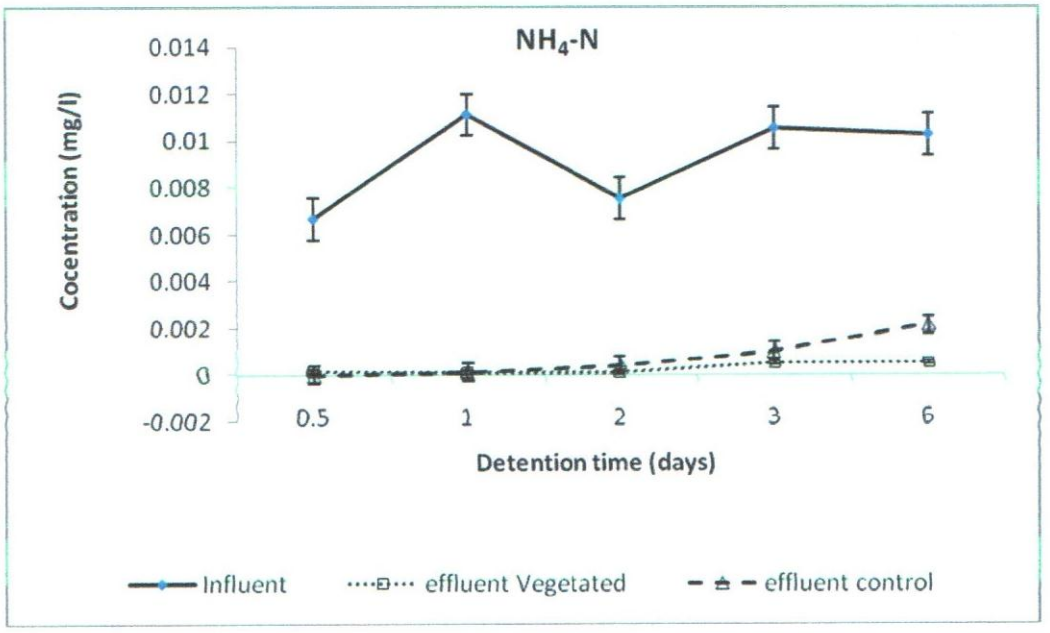


Figure 7: NH_4-N Concentration for influent and effluent from the system at different retention times

Vertical bars = ± Standard errors

With regard to nitrification, the results indicate that approximately 97% and 92% of NH_4-N concentrations is reduced in vegetated and control systems. The plant uptake could be responsible for 5% up take of ammonium. Table 4-10 shows removal efficiencies of the ammonium for different hydraulic loading rates. The difference in removal efficiencies for vegetated and control systems under different hydraulic retention times of 0.5, 1, 2, 3 and 6 days were 2.51%, 0.46%, 3.89% 4.72% and 15.86% respectively. An increase in the difference in treatment efficiencies is noted as retention time increases. This could be attributed to reduced DO levels within the control system thus reduced nitrification processes taking place and as a result higher concentration of ammonium in the effluent.

The vegetated CW performed better in the reduction of TP, SRP and NH_4-N than the control system. As retention time reduced the nutrients concentration also decreased. This could have been as a result of reduced contact time with the substrate and also time for plant uptake of the pollutants. The vegetated system had higher removal efficiency than the control system for TP and SRP, through out the analysis period at the different hydraulic loading rates that were applied (Figure 8 and Figure 9).

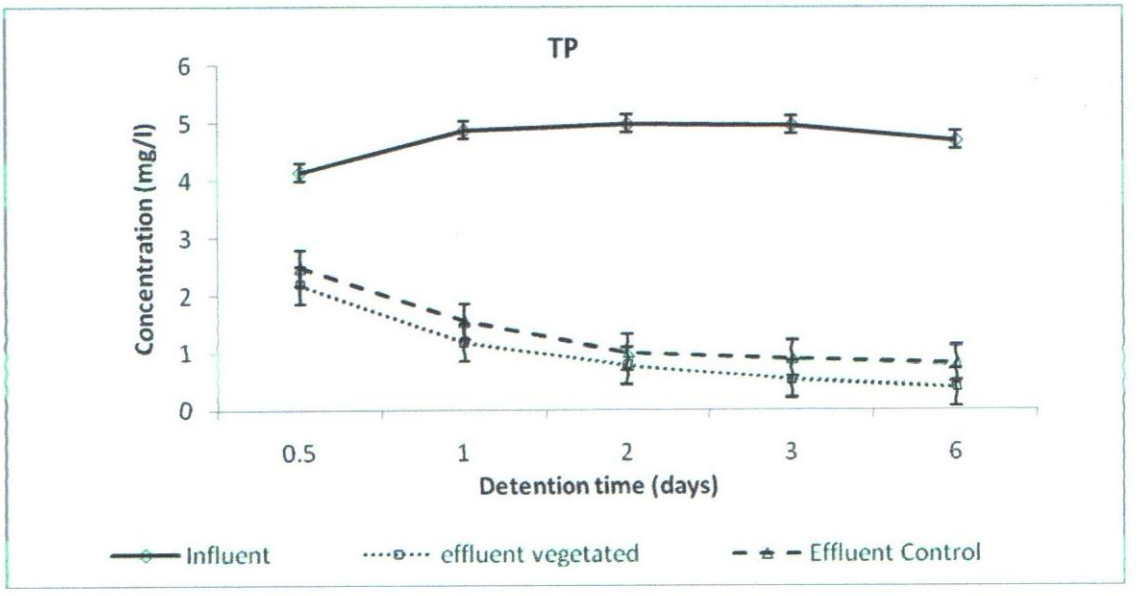


Figure 8: TP concentration for influent and effluent of the system at different retention times

Vertical bars = ± Standard errors

This indicated that vetiver grass played a role in the reduction of these nutrients through filtration process of nutrients and sediments by the plants.

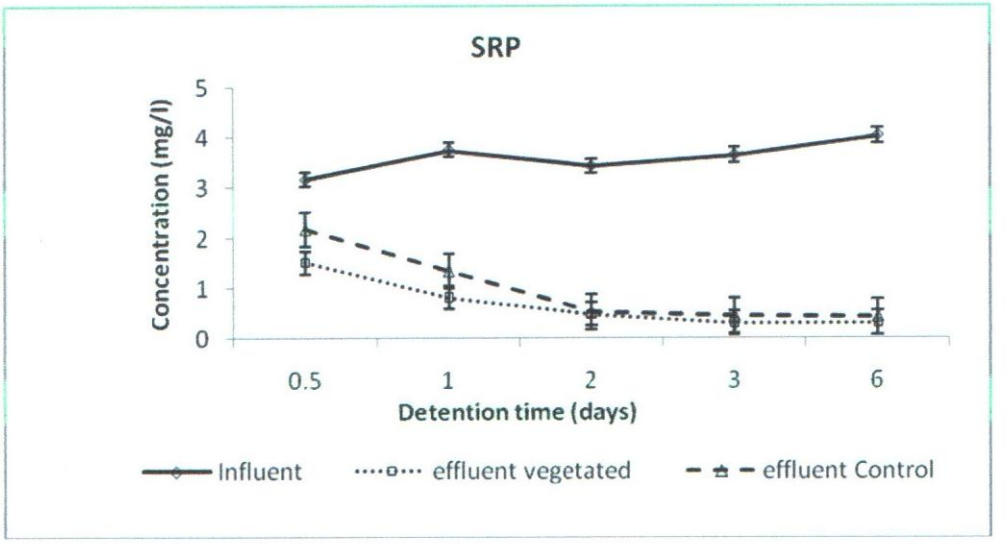


Figure 9: SRP Concentration for influent and effluent of the system at different retention times

Vertical bars = ± Standard deviation

As the loading rates decreased (i.e. increase of retention time) higher removal efficiencies were recorded. This could be attributed to increased contact time of the wastewater with the substrate. The difference in TP removal efficiencies for vegetated and control systems at retention times of 0.5, 1, 2, 3 and 6 days was 7.02, 7.55, 8.37, 7.47 and 8.61% respectively

(Table 4-10). This indicated that adsorption of phosphorous by vetiver grass was not dependent on the retention time. SRP was also noted to reduce with increase in retention time. This implied that vetiver grass played a role in reduction of this nutrient. The SRP reduction by vetiver grass ranges between 4-22%. Generally, phosphate retention in the constructed wetlands depends upon the composition of the wastewater, loading rate, type of root media and the type of substrate used (Brix *et al.*, 1998). Removal of phosphorus in CWs is not temperature-dependent. The most important removal mechanisms are chemical precipitation and physico-chemical sorption, processes that are not temperature-dependent. 5–10% of the overall TP reduction could be attributed to plant uptake (Table 4-10). Phosphate removal therefore was mainly due to retention by the filter bed material. Also the periphyton, i.e. "biofilm" formed of organisms and their remains onto the CW soil and macrophytes, may have enhanced phosphorous retention. According to Dodds (2003), periphyton plays several roles in reduction of phosphorous from the water column. These roles include phosphorous uptake and deposition, filtering particulate phosphorous from the water and increasing oxygen concentrations near the sediment surface thus encouraging the adsorption of dissolved phosphorous.

NO_3-N concentration, within the vegetated and control system was found to increase in all hydraulic loading rates operated. The concentration of this nutrient was varying with the concentration of oxygen within the system. The transfer of oxygen from the atmosphere to the root zone could be a factor causing the increase of NO_3-N tested in the effluent of the vegetated systems. NO_2-N increased as retention time increased in the control system. This could be attributed to reduced DO levels within the control system thus the conversion could not get to NO_3-N and as a result NO_3-N in control system was lower. However, the effluent concentrations of NO_2–N and NO_3–N in all hydraulic loading rates operated were below the NEMA discharge standards of 2 and 50 mg/L, respectively. The decrease in NH_4-N concentration in the effluent suggests that CW systems are effective in reduction of ammonium nitrogen. It was noted that under all hydraulic loading rates applied, NH_4-N concentration of the effluent was lower than the recommended discharge standard by NEMA of 1mg/l.

The results further showed a significant reduction in feacal coliforms concentration in all the hydraulic loading rates applied (Table 4-8). The effluent from the vegetated system had far

much lower concentration of faecal coliforms than the recommended effluent discharge standards of 400 MPN/100ml by NEMA.

4.5 PREWet Model Performance

4.5.1 Model calibration

The PREWet model was applied on two assumptions, the plug flow and the fully mixed flow conditions. The temperature correction factor for BOD and coliform in the plug flow condition was taken to be 1.013 and 1.03 respectively while for fully mixed condition the temperature correction factor for the two parameters were 0.13 and 0.03 respectively. The correction factors in the two options were found to be lower than the default values given in the model. The model default values for the temperature correction factor in the two options are 1.07 and 1.047 for coliform and BOD respectively. The fully mixed option correction factor was significantly lower than the recommended values compared to the plug flow condition. Correction factors in plug flow condition were noted to have a difference of 0.034 and 0.04 for BOD and coliform respectively. The decay factors depended on the retention time since this affected the removal efficiency. The default value for BOD decay factor in the model was 0.8 but this was found to change with retention time as shown in Table 4-11.

Table 4-11: Model decay rate factors at different retention times for the modelled parameters

Parameter	Retention time (days)				
	0.5	1	2	3	6
TSS	0.725	0.456	0.703	0.0633	0.03489
TCB	3.706	1.334	1.997	0.8954	0.585
BOD	3.001	1.597	2.93	1.1675	0.535
TP (areal, vtp)	0.688	0.523	0.782	0.0778	0.04085
TP (volumetric, ktp)	1.229	0.934	1.396	0.695	0.3648

4.5.2 Sensitivity analysis

Changing retention time and fixing wetland dimensions, the temperature correction factor and decay/ degradation coefficients had an influence on the pollutant removal. As hydraulic retention time varied from 1-6 days, the removal efficiency and the outflow concentration of the constituent variables was changing (Figure 10).

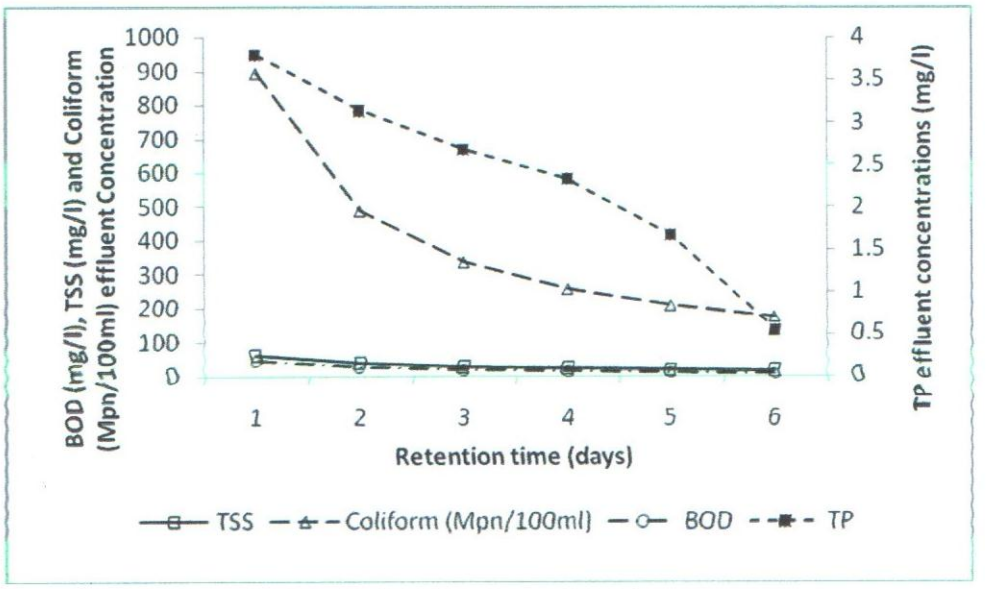


Figure 10: Effect of changing retention time on PREWet model output (plug flow option)

With increasing hydraulic retention time, the pollutants treatment improved significantly (Table 4-12). Similar findings were reported by Zidan *et al.* (2006). Retention time could be increased by changing the wetland volume and applying the same flow rate or by decreasing wetland discharge.

Table 4-12: Effects of changing hydraulic retention time on PREWet model output (fully mixed option)

pollutant	Retention time (days)					
	1	2	3	4	5	6
TSS (mg/l)	50.38	35.25	24.66	17.25	12.07	8.45
Coliform (mpn/100ml)	675.4	380.2	214	120.4	67.8	38.16
BOD (mg/l)	98.51	57.93	34.07	20.04	11.78	8.96
TP (mg/l)	3.10	2.67	1.5	1.04	0.72	0.5

The PREWet model was also found to be sensitive to the depth of water within the wetland system. When the model was operated in fully mixed option, increase in water depth reduced the removal efficiency of TSS and TP while decrease in water depth increased removal efficiency of TSS and TP. This variation could be as a result of reduced contact with the substrate media used in the wetland. The coliform and BOD_5 outflow concentrations were not affected by change in depth of water in the wetland (Table 4-13).

Table 4-13: Effects of changing water depth on PREWet model performance (Fully mixed option)

Depth (m)	TSS (mg/l)	TP (mg/l)	Coliform	BOD (mg/l)
Actual values (Depth=0.56)	8.449	0.5009	38.4	6.932
0.4	6.236	0.3696	38.4	6.932
0.46	7.08	0.4199	38.4	6.932
0.5	7.63	0.4527	38.4	6.932
0.6	8.966	0.5324	38.4	6.932
0.66	9.742	0.5788	38.4	6.932
0.76	10.99	0.6536	38.4	6.932

When the water depth within the wetland was changed under plug flow condition, the model results showed that this had an effect on treatment performance on the TSS while the other modelled constituents were not affected (Table 4-14).

Table 4-14: Effects of changing water depth on PREWet model Performance (Plug flow condition)

Depth	TSS (mg/l)	TP (mg/l)	TCB	BOD (mg/l)
Actual values (Depth=0.56)	8.447	0.5005	38.16	6.929
0.46	5.301	0.5005	38.16	6.929
0.5	6.532	0.5005	38.16	6.929
0.6	9.744	0.5005	38.16	6.929
0.66	11.69	0.5005	38.16	6.929

When the retention time of the wetland was maintained constant and the volume of the wetland changed, the removal efficiency and outflow concentration were not affected. It was also noted that changing width of the wetland and maintaining hydraulic retention time had no effect on out flow concentration of the constituent.

4.5.3 Model simulation

The coefficients of correlation for the observed and simulated parameters from the model were determined. The parameters included TP, faecal coliforms, BOD and TSS with coefficient of correlation of 0.985, 0.981, 0.983 and 0.901 respectively. A scatter plot diagram developed for modeled constituent together with the line of best fit drawn and coefficient of determination (r^2) are given in Figures 11, 12, 13 and 14. The r^2 for the TP and coliform was 0.971 and 0.961 respectively.

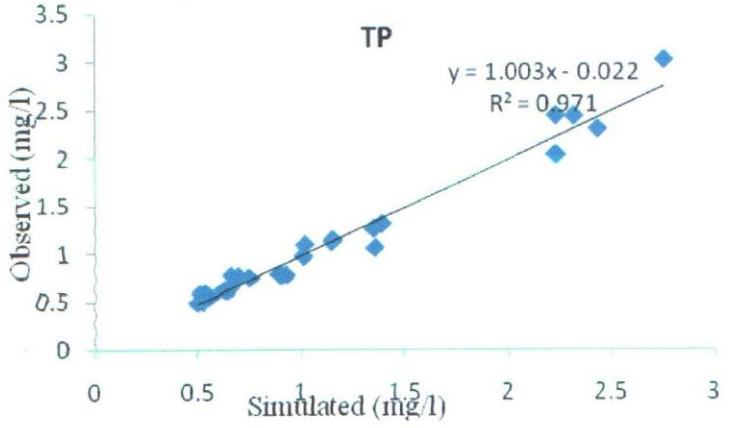


Figure 11: The r^2 of observed and simulated concentrations of TP

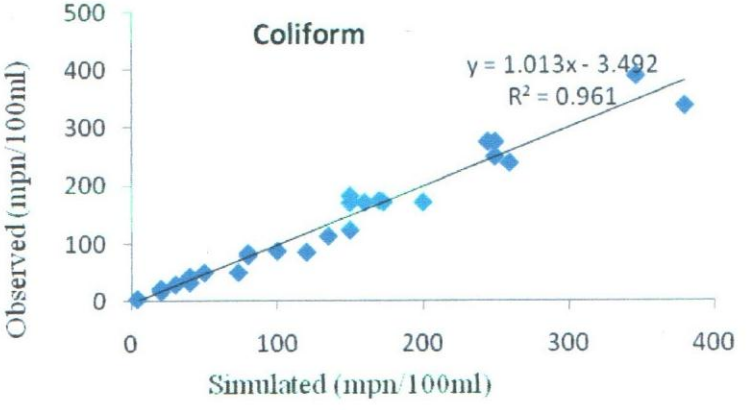


Figure 12: The r^2 of observed and simulated concentrations of Coliform

The r^2 for the BOD_5 and TSS was calculated to be 0.966 and 0.81 respectively as given in Figures 13 and 14.

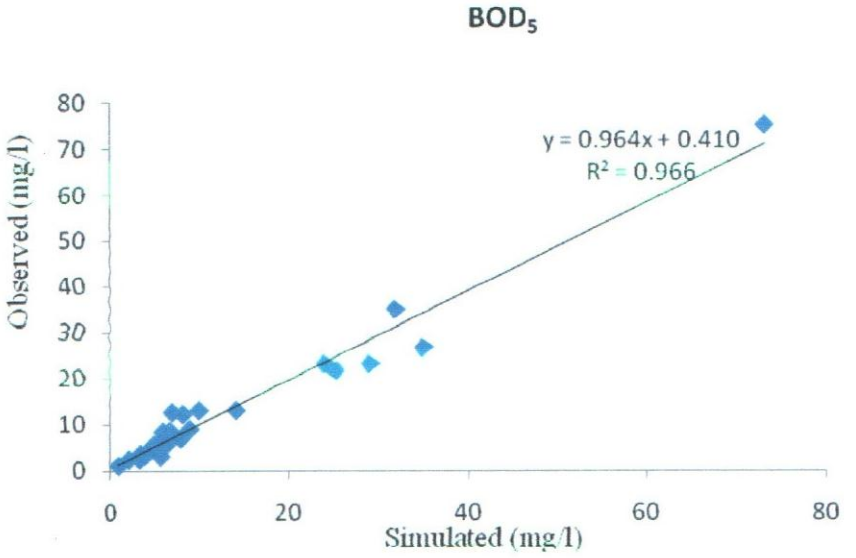


Figure 13: The r^2 of observed and simulated concentrations of BOD_5

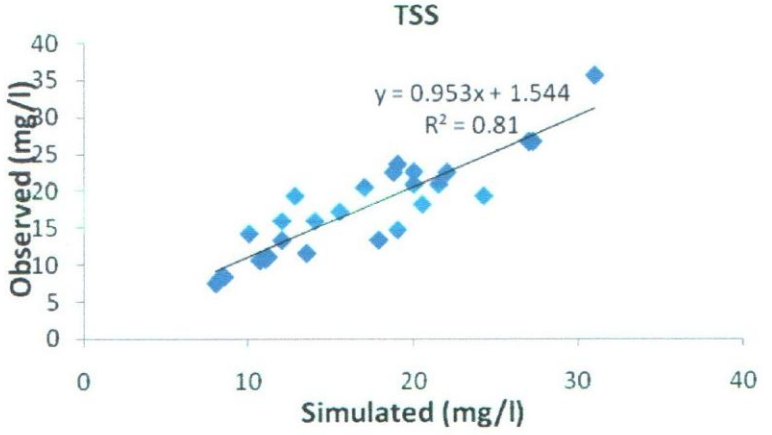


Figure 14: The r^2 of observed and simulated concentrations of TSS

The calculated Nash–Sutcliffe coefficient for BOD_5, TP, Coliform and TSS were 0.97, 0.97, 0.96 and 0.77 respectively. According to Moriasi et al. (2007), the values of Nash–Sutcliffe coefficient lower than 0.5 indicates that the model should be re-calibrated. In this study, simulated and observed parameters of all the constituents modelled had a strong agreement and indicated that the model can be satisfactorily used in prediction of wastewater treatment through constructed wetland(Figures 15, 16, 17 and 18).

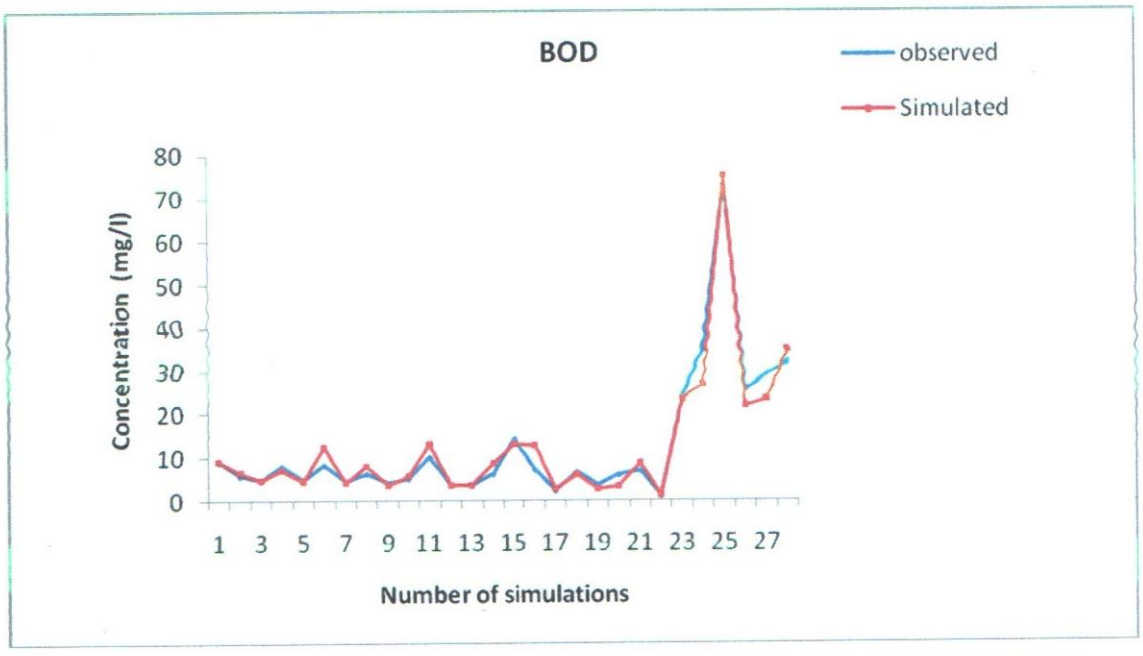


Figure 15: The observed against the simulated concentrations of BOD_5

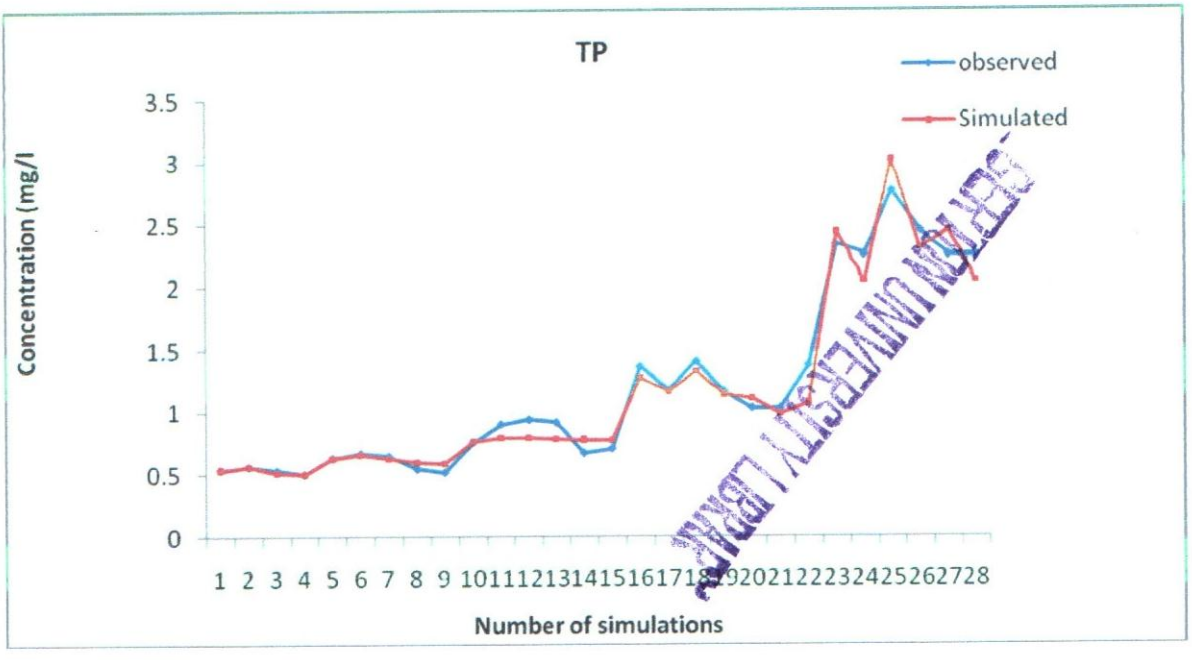


Figure 16: The observed against the simulated concentrations of TP

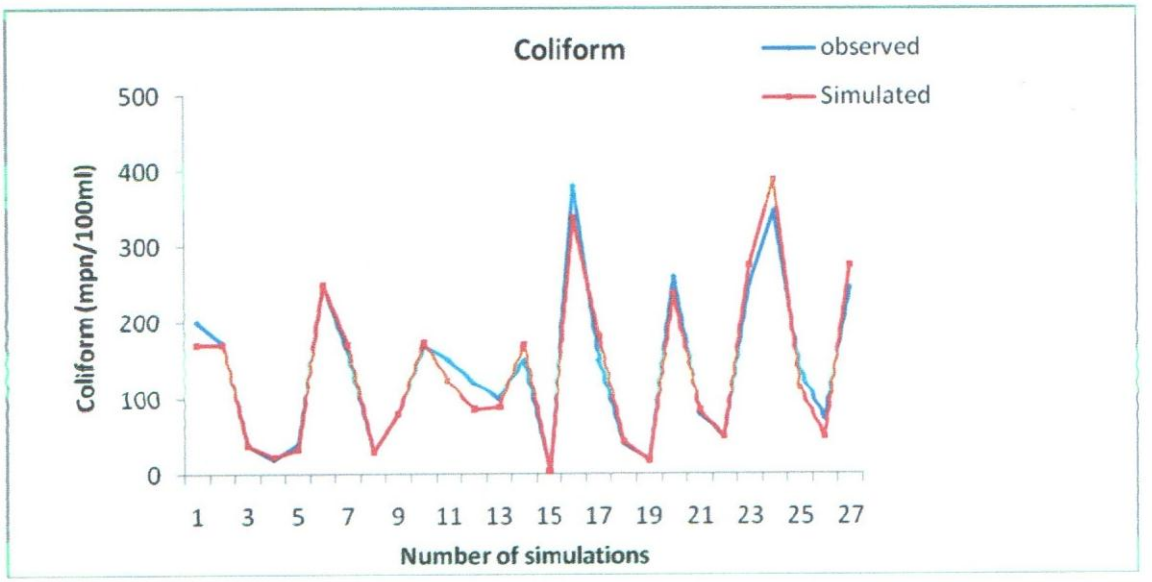


Figure 17: The observed against the simulated concentrations of coliform

Both coefficient of correlation and Nash-Sutcliffe coefficient indicated that TSS had the lowest agreement between the observed and simulated values.

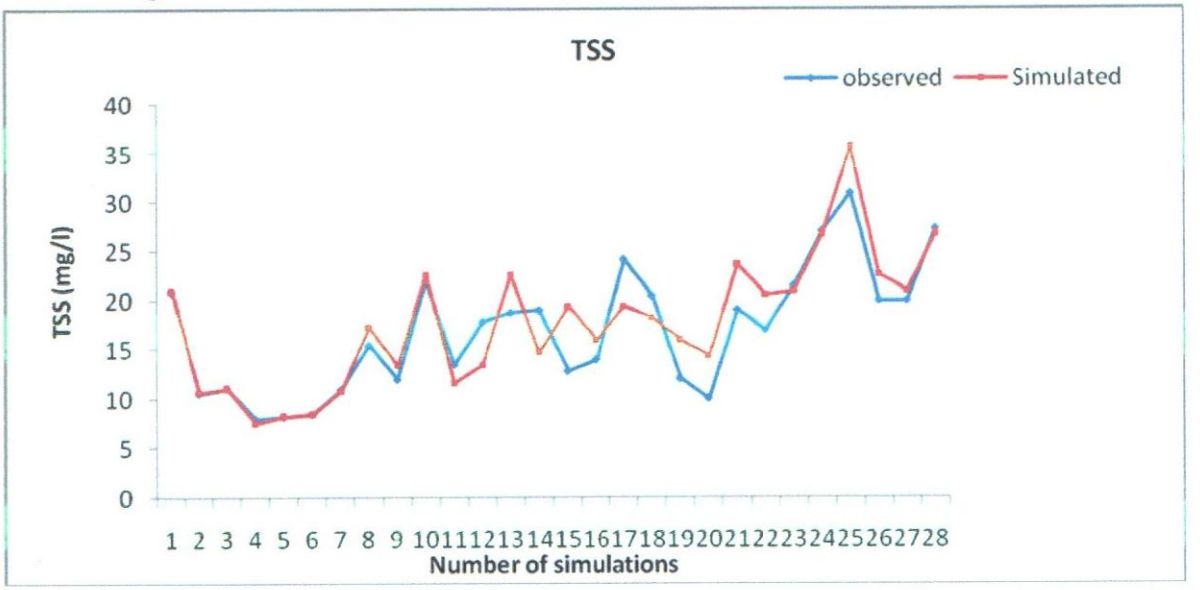


Figure 18: The observed against the simulated concentrations of TSS

4.5.4 Model prediction

Wastewater data collected from Nakuru (Kaloleni) wastewater stabilisation ponds had mean concentration of TP, TSS, BOD and coliform of 5191 µg/l, 20.49 mg/l, 142.63 mg/l and 1350 MPN/100ml respectively (Appendix G). Tables 4-15 and 4-16 give the concentration of various variables in Nakuru (Kaloleni) wastewater from the PREWet model at different

retention times. The plug flow condition and fully mixed option predicted the treatment of the wastewater uniformly in all the retention times applied.

Table 4-15: Model prediction on Nakuru data using plug flow condition

HRT (days)	TSS (mg/l)	TP (mg/l)	BOD (mg/l)	Coliform (MPN/100ml)
0.5	10.84	3.199	36.1	210.5
1	5.899	1.461	8.739	182.8
2	4.062	0.9116	6.725	94.54
3	3.788	0.7337	4.944	92.86
6	3.193	0.6616	6.619	41.34

Fully mixed option had an under estimation of TSS at all hydraulic retention time operated but all the other modelled constituents were predicted right (Table 4-16).

Table 4-16: Model prediction on Nakuru data using fully mixed condition

HRT (days)	TSS (mg/l)	TP (mg/l)	BOD (mg/l)	Coliform (MPN/100ml)
0.5	10.8	3.199	36.09	210.3
1	5.699	1.521	8.737	183.3
2	3.962	0.9095	6.726	94.61
3	3.725	0.734	6.622	92.87
6	2.426	0.6618	5.094	41.6

It was noted that for the effluent from MCN (Kaloleni) wastewater stabilisation pond to be effectively polished using a CW, the effluent from the ponds requires a retention time of 2 days. At retention time of 2 days all the constituents modelled met the NEMA and WRMA discharge standards.

PREWet model considers that the wetland vegetations and plants growth have no effect on reduction of phosphorous since the plants litter drops naturally at the wetland bottom. The litter decays through microbial processes with the plant litter therefore acting as a source of TP and absorbed contaminants.

PREWet model was developed in US which experiences winter and during this cold season plants easily die off thus releasing the nutrients back into the wetland water. However, the operating condition of the pilot wetland was different from that of the PREWet model site specific conditions. This is because temperature in Njoro Kenya is not too low to cause

wetland plants death. Vetiver grass is a hardy plant which does not fall off easily and after its growth it can be harvested easily and fed to animals. When the grass is harvested it gets rid of the stored pollutants at plants parts thus enhancing plant growth which will encourage more nutrient uptake. In this regard effect of vetiver grass in reduction of nutrients especially phosphorous was included during the calibration process of the PREWet model.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

As human settlement becomes more and more centralised, production and disposal of large amount of wastewater is becoming a major environmental problem. Though previously such wastewater used to be released into large water bodies and rivers without any treatment with the rivers' self purification processes, purifying such wastes, it has become necessary that wastewater treatment be done due to the large volumes of wastewater produced. Though Egerton University wastewater treatment plant produces effluent with improved quality, its organic matter content is still high thereby requiring a longer treatment for degradation of high BOD_5. Therefore, one of the main areas of concern in this study was to lower the organic matter content of the effluent from Wastewater Stabilization Ponds (WSPs) using a vertical flow constructed wetland.

This study examined a number of parameters of the vertical flow constructed wetland and found out that the major parameter that influences the operation of wetlands in the polishing of wastewater is its Hydraulic Retention Time (HRT). In this case therefore, the study established that HRT is an important parameter in sizing of constructed wetlands. To adequately polish the effluent from Egerton University's WSPs, the HRT of 2.1 days within the constructed wetland was found to be effective. In addition, the study found out that other factors such as the depth of the water within the wetland system, porosity, hydraulic conductivity and the type of the substrate used in Constructed Wetlands (CWs) have great effects on the polishing of effluent from WSPs. Therefore, when designing a CW, it is important that such factors are strongly considered in order to develop a well functioning system.

The CW reduced nutrients and microbial pollutants in the influent in which case the reduction was varying with different Hydraulic Loading Rates (HLR) applied. At low loading rates, a higher removal efficiency of pollutants was recorded while the reduction deteriorated with an increase of HLR. The HLR applied ranged between 0.014 - 0.174 m/day and the removal efficiency of all nutrients analysed was between 97-47%. In all the HLRs applied, the effluent concentration of coliform in the CW was lower than the recommended effluent discharge standards of 400 MPN/100ml by National Environmental Management Authority (NEMA). In this study also, the expanded clay was found to be effective in the reduction of the

nutrients especially TP and SRP in wastewater with the reduction ranging between 90-40% and 91-47% respectively.

Although the variation of the depth of water in the wetland had an effect on the performance of the PREWet model in varying levels of concentration of TSS and TP, the main governing parameter in PREWet model application was found to be the hydraulic retention time. The physical dimensions (length, width, or height) were found to have minimal effects on performance of the model. The study found out that the results simulated using the PREWet model and those observed or measured matched closely. The coefficient of determination for the simulated and observed values for the modeled constituents ranged between 0.81–0.97. In addition, the coefficient of correlation of the modelled constituents between the observed and simulated was between 0.90-0.99. Using the statistical Nash–Sutcliffe method, the observed and the simulated coefficient of model efficiency ranged between 0.77–0.97. From this study it can be concluded that the PREWet model can be useful in planning of wastewater treatment when the data for influent characteristics and the volume of wastewater to be polished are available.

5.2 Recommendations

The results from this study provide a baseline for understanding the performance of the constructed wetland in polishing of effluent from wastewater stabilization ponds and will be used as a basis for future studies. Based on this study the following recommendations are made for further improvement of wastewater treatment;

1. Since the depth of the substrate can affect the treatment efficiency of the constructed wetland, there is need to find out the extent to which the varying depth of substrate media in a wetland will affect the removal efficiency of the pollutants.
2. Some research need to be undertaken to establish whether different mixing ratio for sawdust and clay in preparation of expanded clay would have a significant effect on the treatment efficiency of the wetland. Such a research may also be based on whether mixing clay with another media other than sawdust has an impact in nutrients adsorption capacity.

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APPENDICES

Appendix A: Formulae for calculating self purification ability.....72

Appendix B: Laboratory analysis

Appendix B-1: Nutrient analysis.....74

Appendix B-2: Winkler method for determination of dissolved oxygen.....78

Appendix B-3: MPN Table.....80

Appendix C: Pilot system set up.....82

Appendix D: Results from Municipal Council of Nakuru (Kaloleni) WSPs.....84

Appendix E: PREWet model

Appendix E-1: Model constants and removal efficiencies.....85

Appendix E-2: Model simulation results.....86

Appendix F: Mean nutrient concentration of influent and effluent from the vegetated and control systems for different retention times 87

Appendix G: Nutrients concentration for the different sampling sites along Njoro River from October 2009 to February 2010.....88

APPENDIX A

FORMULAE FOR CALCULATING SELF PURIFICATION ABILTY

The rate of addition of BOD by local runoff and by product of anaerobic digestion of bottom sludge deposits L_c can be determined from the following relationship,

$$Lx = La \exp^{\left(-K1\frac{x}{v}\right)} \quad \text{(A-1)}$$

Where L_x = estimated value of BOD_L at a distance x from the point of discharge

L_a = initial BOD_L or BOD at point of discharge

x = distance, m

v = velocity of flow.

If L_x is greater than the actual value of L, it is assumed that $L_c = 0$ and thus K_3 is estimated from Equation 2

$$L = L_a \exp\left[-(K_1+K_3)\frac{x}{v}\right] \quad \text{(A-2)}$$

Where K_3 = dispersion coefficient.

If L_x is less than the actual value of L, it is assumed that $K_3 = 0$ and the value of L_c is calculated using the equation below substituting $K_3 = 0$.

$$L = La \exp\left(-K_1\frac{x}{v}\right) + \left(\frac{Lc}{K_1}\right)\left[1 - \exp\left(-K_1\frac{x}{v}\right)\right] \quad \text{(A-3)}$$

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If L_x is equal to the actual value of L, both $K3$ and L_c may be assumed to be equal to zero.

Sr is the rate of change of DO concentration as a result of physical and biological forces. This can be determined from the Equation 4

$$Dx = Da\,F_2 + \frac{K_1}{K_2-(K_1+K_3)}\left[La\frac{L_b}{K_1+K_3}\right](F_1+F_2) + \left[\frac{Sr}{K_2} + \frac{K_1\,Lb}{K_2(K_1+K_3)}\right](1-F_2) \quad \text{(A-4)}$$

Da is DO at $x = 0$ or initial DO and Dx is the DO at distance x

$$F_2 = \exp\left(-K_2\frac{x}{v}\right) \quad \text{(A-5)}$$

$$F_1 = \exp\left[-(K_1+K_3)\frac{x}{v}\right] \quad \text{(A-6)}$$

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$$F_3 = \frac{Sr}{K_2} + \frac{K_1 Lc}{K_2(K_1+K_2)} \tag{A-7}$$

$$F_4 = \frac{K_1}{K_2 - (K_1 + K_3)} \tag{A-8}$$

$$D = \frac{k_1 la}{k_2 - k_1}\left[\exp\left(-k_1\frac{x}{v}\right) - \exp\left(-k_2\frac{x}{v}\right)\right] + D_a \exp\left(-k_2\frac{x}{v}\right) \tag{A-9}$$

APPENDIX B

LABORATORY ANALYSIS

1. Nutrient analysis

Soluble reactive phosphorus (SRP)

Soluble reactive phosphorus (SRP) was determined using the ascorbic acid method (APHA, 1995) on the filtered samples. All the glassware used was acid-washed with 10% H_2SO_4 at least one day before the commencement of the exercise and rinsed with distilled water.

Standard calibration curve

Standard solution for the calibration of the standard curve was made by dissolving 5.623 g of potassium hydrogen phosphate (K_2HPO_4) pre-dried salt in the oven for 24 hrs at approximately 70°C in 1 litre of distilled water to make a stock solution with a concentration of 1g of PO_4-P l^{-1}. From this stock solution, 10 ml was diluted upto 1 litre using distilled water to give an intermediate solution with a concentration of 10 mg l^{-1}. The intermediate solution was further diluted in a ratio of 1:20 (25 ml in 500 ml-distilled water) to give a working solution with a concentration of 500 µg l^{-1}. The standard series was prepared by taking the following respective volumes of the working solution and diluting to 25 ml with distilled water as shown on the table below. Triplicate samples were taken for each concentration.

Table B-1: Standard series concentration of SRP working solution

Conc. (µg l^{-1})	0	10	20	50	100	200	400	500
Working solution (ml)	0	0.5	1.0	2.5	5.0	10.0	20.0	25.0
Vol. dist. water (ml)	25.0	24.5	24.0	22.5	20.0	15.0	5.0	0.0

Reagents:

A) Ammonium molybdate solution.

15 g $(NH_4)6MO_7O_{24}$ x 4 H2O) in 500 ml distilled water

B) Sulphuric acid:

140 ml concentrated sulphuric acid was diluted up to 1000 ml with distilled water

C) Ascorbic acid:

2.7 g ascorbic acid was dissolved in 50 ml distilled water. This was always freshly prepared since the acid is unstable and must be used immediately before 24 hrs.

D) Potassium-Antimonyltartrate-Solution:

0.34 g K-Antimonyltartrat was dissolved in 250 ml distilled water

The four reagents were mixed in the respective ratios: A: B: C: D = 2:5:2:1, where the order of

mixing is very critical. The resulting solution was added to the sample in a ratio of 1:10, i.e. 2.5 ml reagent added to 25 ml of the diluted sample. The prepared sample's absorbance was measured after 15 minutes of adding reagents to the samples at a wavelength of **885 nm** with distilled water as a reference.

Total Phosphorus

Total phosphorus was determined by first digesting and reducing the forms of phosphorus present in the water into the free ortho-phosphate form (SRP) using persulphate digestion. After digestion, the total phosphorous reduced forms into SRP formed was analysed using the same procedure as for the soluble reactive phosphorus. A standard calibration curve for total phosphorus was prepared in which the standard solutions underwent the persulphate digestion procedure followed with the soluble reactive phosphorus test procedure.

Reagents:

Potassium persulphate $K_2S_2O_8$

12g of $K_2S_2O_8$ was dissolved in 100 ml of distilled water by sonification for about half an hour. To 25 ml of dilute sample, 1 ml of the still warm $K_2S_2O_8$ solution was added. The bottles were weighed without the lids and their weight noted. The lids were put back but not closed tightly after which they are autoclaved for 90 minutes at about 120°C. After cooling, the bottles were re-weighed and the evaporated water replaced by addition of distilled water. 25 ml of diluted sample was used for determination of total phosphorus and for this the steps used for the soluble reactive phosphorus should was used.

Nitrate-nitrogen (NO_3-N)

Nitrate-nitrogen was determined using sodium-salycilate method (APHA, 1995) with standard solutions of the nitrate prepared for the standard calibration curve. Fresh sodium salicylate solution was always used in each determination.

Standard calibration curve

A stock solution was prepared by dissolving 6.067g of sodium nitrate ($NaNO_3$ p. a) in 1 litre of distilled water to make a stock solution of 1000 mg l^{-1}. From this stock solution, a working solution with a concentration of 2.5 mg NO_3-N l^{-1} was made by taking 5 ml of the stock solution and diluting to 1 litre. A standard series with different concentrations was then made by taking the given concentrations of the working solution and diluting with distilled water as shown on the table below.

Table B-2: Standard series concentration of NO_3-N working solution

Conc. (mg l^{-1})	0	0.25	0.5	1	2.5	5
Working soln (ml)	0	1.0	2.0	4.0	10.0	20.0
Vol. dist. Water (ml)	20	19.0	18.0	16.0	10.0	0.0

Triplicate samples for each concentration was made to make the calibration curve.

Reagents:

A) Sodium salicylate-solution:

This solution was freshly prepared since it is stable for only one day by dissolving 0.5 g Na-Salicylate in 100 ml distilled water.

B) Concentrated sulphuric acid

C) Potassium-sodium tartarate solution:

400 g NaOH was dissolved in 1 litre distilled water, together with 50 g K-Na-Tartrate to make this solution.

20 ml of filtered water sample was placed in an evaporation bottle and to this as well as to the standard series, 1 ml of sodium salycilate solution freshly prepared was added. The bottles were then put into the oven and the samples dried at a temperature of 80°C. The resulting residue was dissolved quantitatively by adding 1 ml of conc. H_2SO_4 and the bottles swirled carefully while still warm. Next, 40ml of distilled water was added and mixed. Finally 7 ml of potassium-sodium hydroxide-tartarate solution was added mixed and the absorbance determined at a wavelength of **420 nm**.

Nitrite-Nitrogen ($N0_2$-N)

The nitrite-nitrogen determination was carried out using the reaction between sulfanilamid and N-Naphthyl-(1)-ethylendiamin-dihydrochlorid which gives an intense pink colour with the nitrite.

Standard calibration curve

$NaNO_2$ salt with a molar weight of 69 g was used to make the standard calibration curve for the nitrite determination. A stock solution with a concentration of 1 g NO_2-N L^{-1}, was made by dissolving 1.2322 g of this salt in 250 mls distilled water. 5 ml of this stock solution was diluted to 500 ml with distilled water to make an intermediate solution having a concentration 10 mg L^{-1}. 5 ml of this intermediate solution was diluted into 1 litre with distilled water to give a working solution with a concentration of 50 µg L^{-1}. A series of standards with determined concentrations

was made to determine the calibration curve by diluting the working solution as given in the table below.

Table B-3: Standard series concentration of NO_2-N working solution

Conc. ($\mu g\ L^{-1}$)	0	2	5	10	20	50
Working solution (ml)	0	1	2.5	5	10	25
Vol. dist. Water (ml)	25	24	22.5	20	15	0

25 mls of the respective concentrations was taken in triplicate, the test reagents added and the absorbance read from the spectrophotometer to develop the calibration curve.

Reagents:

A: Sulfanilamid solution:

25 ml Conc. HCl (37%) diluted to 150 ml with distilled water. To this dilute acid, 2.5g of sulfanilamide was added. The solution was then further diluted upto 250 ml with distilled water.

B: N-Naphthyl-(1)-ethylendiamin-dihydrochlorid solution.

0.25g of N-Naphthyl-(1)-ethylendiamin-dihydrochlorid was mixed in 250 ml with distilled water.

To 25 ml of the filtered sample, 1 ml of solution A was added. After 2-8 minutes, 1 ml of solution B was added to this mixture and gently mixed. The solution was left standing for 10 minutes after which its absorbance was read from the spectrophotometer at a wavelength of **543 nm**. The colour complex formed is normally stable for two hours.

Ammonium-nitrogen (NH_4-N)

Standard calibration curve

This was made by using NH_4Cl solids with a molecular weight of 53.492 g in which the nitrogen proportion of NH_4–N was 1:14 of the weight. 0.955 g NH_4Cl was dissolved in a 250 ml volumetric flask to prepare 250 ml of this solution, giving a stock solution with a concentration of 1g NH_4-N l^{-1}. 10 ml of this stock solution was diluted further to 1 litre with distilled water, giving an intermediate solution with a concentration of 10 mg l^{-1}. From this solution, a working solution was made with a concentration of 250 $\mu g\ l^{-1}$ by taking 25 mls of the intermediate solution and diluting it upto 1 litre. The working volume for calibration curve used was 25 ml.

Different concentrations of this standard working solution was made to determine the calibration curve by taking the following respective volumes of the working solution and diluting to 25 ml with distilled water as given in the table below.

Table B-4: Different concentration of NH_4-N standard working solution

Conc. ($\mu g\ l^{-1}$)	0	10	20	50	100	250
Working solution (ml)	0	1.0	2.0	5.0	10.0	25.0
Vol. dist. water (ml)	25	24.0	23.0	20.0	15.0	0.0

A triplicate sample for each of the concentrations was used to determine the calibration curve.

Reagents

A) Sodium salicylate solution

130 g of sodium salicylate and 130 g of trisodium–dihydrate was mixed in 800 ml of distilled water. 0.97 g of sodium nitroprussid was then added to this solution. The solution was filled upto 1000 ml using distilled water.

B) Hypochlorid solution

0.2g of sodium dichloroisocynurate was freshly prepared by adding and mixing 100 mls of NaOH solution. The NaOH solution was made by dissolving 32 g NaOH in 1000 ml distilled water was prepared early and has a bench life of two months.

Procedure

To 25 mls of diluted sample, 2.5 mls of reagent A was added, followed immediately by addition of 2.5 mls of reagent B. The sample was then placed in a water bath at a temperature of 20°C in the dark for 90 minutes. Absorbance was then determined at a wavelength of **655 nm**.

2. Winkler method for determination of Dissolved Oxygen

This involves taking a known volume of water in a bottle and oxygen content in the bottles was fixed immediately by adding 1 ml of the Winkler's reagent I ($MnSO_4$ solution) and the same volume of Winkler's reagent II (KI solution) below the surface of the samples in the bottles and the bottles closed and shaken vigorously. 1 ml of conc. H_2SO_4 was added to the samples and the bottles shaken to dissolve the precipitate. The solution was transferred into Erlenmeyer flask and titrated with 0.01 N sodium thiosulphate until a faint yellow colour appears. 1 ml of starch

indicator was added to the sample and the titration continued until the blue colour of the solution disappeared marking the end point. The amount of dissolved oxygen was calculated using the formula by Wetzel & Likens (1991):

$$O_2\ conc(mg/l) = ml of titrant \times (Molarity\ of\ thiosulphate)\ (8000) \div (Volume\ titrated \times Vol\ of bottle - 1)$$

The factor 8000 in the formula comes from the fact that 1 ml of N sodium thiosulphate is equivalent to 8 mg O_2 and the conversion of concentration per litre.

B-5: MPN Table

Number of positive tubes			MPN per 100ml
1ml	0.1ml	0.01ml	
0	0	0	0
0	0	0	20
0	1	0	20
1	0	0	20
1	0	1	40
1	1	0	40
1	2	0	50
2	0	0	40
2	0	1	50
2	1	0	50
2	1	1	70
2	2	0	70
2	3	0	110
3	0	0	70
3	0	1	90
3	1	0	90
3	1	1	130
3	2	0	130
3	2	1	160
3	3	0	160
4	0	0	110
4	0	1	140
4	1	0	160
4	1	1	200
4	2	0	200
4	2	1	250
4	3	0	250
4	3	1	310
4	4	0	320

Number of positive tubes			MPN per 100ml
1ml	0.1ml	0.01ml	
4	4	1	380
5	0	0	220
5	0	1	290
5	0	2	410
5	1	0	310
5	1	1	430
5	1	2	600
5	1	3	850
5	2	0	500
5	2	1	700
5	2	2	950
5	2	3	1200
5	3	0	750
5	3	1	1100
5	3	2	1400
5	3	3	1750
5	3	4	2100
5	4	0	1300
5	4	1	1700
5	4	2	2200
5	4	3	2800
5	4	4	3450
5	5	0	2400
5	5	1	3500
5	5	2	5400
5	5	3	9100
5	5	4	16000
5	5	5	>18000

Source:APHA,1995.

APPENDIX C
PILOT SYSTEM SET UP

Table C-1: Hydraulic Conductivity (ks) for expanded clay

S/N	H (cm)	Time (sec)	Outflow (cm3)	Q (cm3/sec)	L (Cm)	A (cm2)	ks (cm/sec)	ks (m/day)
1	57	600	417	0.7	16	201.09	0.0009702	0.84
2	57	600	410	0.68	16	201.09	0.0009539	0.82
3	57	600	390	0.65	16	201.09	0.0009073	0.78
4	57	600	370	0.62	16	201.09	0.0008608	0.74
5	57	990	570	0.58	16	201.09	0.0008037	0.69
6	57	900	490	0.54	16	201.09	0.00076	0.66
Avg	57	715	441.17	0.63	16	201.09	0.000876	0.76

Table C-2: Hydarulic conductivity (ks) for sand

S/N	H (cm)	Time (sec)	Outflow (cm3)	Q (cm3/sec)	L (Cm)	A (cm2)	ks (cm/sec)	ks (m/day)
1	48	900	410	0.46	16	201.088	0.0007552	0.65
2	48	1020	470	0.46	16	201.088	0.0007638	0.66
3	48	600	290	0.48	16	201.088	0.0008012	0.69
Avg	48	840	390	0.47	16	201.088	0.0007734	0.67

Table C-3: Water budget

temp	As	P (m/day)	ET (m/day)	P (V/t)	**ET** (V/t)	Qi (V/t)	Qe (v/t)	Dv/dt
20.09	0.83	0.0059	0.0032	0.0049	0.0026	0.0130	0.0110	0.0043
20.95	0.83	0.0111	0.0035	0.0092	0.0029	0.0340	0.0380	0.0023
19.86	0.83	0.0124	0.0034	0.0103	0.0028	0.0140	0.0100	0.0115
20.68	0.83	0.0079	0.0031	0.0066	0.0026	0.0280	0.0200	0.0120
20.07	0.83	0.0145	0.0029	0.0120	0.0024	0.0420	0.0300	0.0216
19.19	0.83	0.0066	0.0025	0.0055	0.0021	0.0740	0.0700	0.0074
19.79	0.83	0.0090	0.0026	0.0074	0.0022	0.1480	0.1400	0.0133
20.09	0.83	0.0096	0.0030	0.0080	0.0025	0.0260	0.0220	0.0095
19.45	0.83	0.0083	0.0032	0.0068	0.0026	0.0800	0.0640	0.0202
Mean					**0.0025**			**0.0113**

APPENDIX D

RESULTS from Nakuru (Kaloleni) WSPs

Table D-1: physico- chemical parameters of effluent from Nakuru (Kaloleni) WSPs

Parameter	Minimum	Maximum	Mean	Std. Deviation
Oxygen (mg/l)	1.95	3.87	2.71	0.76
Oxygen Saturation (%)	27.40	53.40	37.68	10.21
Temp (°C)	20.30	22.00	21.34	0.65
pH	7.70	7.97	7.82	0.11
EC	0.78	0.92	0.86	0.06
Salinity	0.40	0.50	0.44	0.05
TDS (mg/l)	0.54	0.64	0.59	0.038
Coliform (MPN/100ml)	310.00	2450.00	1350.00	786.23
TSS (mg/l)	16.00	28.50	20.49	5.05
Organic (mg/l)	12.50	16.70	15.23	1.55
Inorganic (mg/l)	.60	11.80	5.39	4.27
BOD (mg/l)	84.00	173.00	142.63	31.16

Table D-2: Nutrients of effluent from Nakuru (Kaloleni) WSPs

Parameter	Minimum	Maximum	Mean	Std. Deviation
TP (mg/l)	4.448	5.902	5.191	0.576
SRP (mg/l)	3.788	4.915	4.438	0.459
NH4-N (mg/l)	10.961	18.621	13.956	3.259
NO3-N (mg/l)	0.117	0.286	0.184	0.067
NO2-N (mg/l)	0.031	0.114	0.067	0.033

APPENDIX E
PREWet Model

Model constants and removal efficiency

TableE-1: TP model constants, removal rate/day and removal efficiency at different detention times

Detention time (days)	Removal rate / day		Decay rate/day		Removal Efficiency (%)	
	Longitudinal	Fully	Longitudinal	Fully	Longitudinal	Fully
0.5	1.223	1.69	0.688	0.1892	45.75	45.8
1	1.396	2.879	0.782	0.3225	75.23	75.22
2	0.9339	2.741	0.523	0.307	84.55	84.57
3	0.695	2.346	0.0778	0.2628	87.57	87.56
6	0.3647	1.32	0.04085	0.1478	88.79	88.79

Table E-2: Coliform model constants, removal rate/day and removal efficiency at different detention times

Detention time (days)	Removal rate/day		Decay rate		Removal Efficiency (%)	
	Longitudinal	fully	Longitudinal	Fully	Longitudinal	Fully
0.5	3.641	10.36	3.706	1.264	83.81	83.82
1	1.962	6.091	1.997	0.743	85.94	85.9
2	1.311	6.37	1.334	0.777	92.73	92.72
3	0.8797	4.333	0.8954	0.5285	92.86	92.86
6	0.5741	5.042	0.585	0.615	96.82	96.8

Table E-3: TSS model constants, removal rate/day and removal efficiency at different detention times

Detention time (days)	Removal rate/day		Decay rate		Removal Efficiency (%)	
	Longitudinal	Fully	longitudinal	Fully	Longitudinal	Fully
0.5	3.641	1.82	0.725	0.2038	47.66	47.64
1	1.962	2.509	0.703	0.281	71.5	71.5
2	1.311	6.071	0.456	0.68	92.38	92.39
3	0.8797	1.471	0.0633	0.16473	81.65	81.52
6	0.5741	0.9125	0.03489	0.1022	84.97	84.56

Table E-4: BOD model constants, removal rate / day and removal efficiency at different detention time

Detention time (days)	Removal rate/day		Decay rate		Removal Efficiency (%)	
	Longitudinal	Fully	Longitudinal	Fully	Longitudinal	Fully
0.5	2.978	6.867	3.001	2.019	77.44	77.44
1	2.907	17.31	2.93	5.09	94.54	94.54
2	1.311	11.39	1.597	3.35	95.8	95.8
3	1.159	10.44	1.1675	3.07	96.82	96.82
6	0.4316	3.86	0.535	1.135	96.91	96.91

1. **Model Simulation**

Table E-5: Simulated and Observed Model parameters

BOD		TSS		Coliform		TP	
observed	Simulated	observed	Simulated	observed	Simulated	observed	Simulated
8.97	8.95	21	20.98	200	171.7	0.53639	0.5362
5.69	6.29	10.64	10.64	172.8	171.7	0.561	0.5597
4.67	4.67	11.17	11.11	38.4	38.16	0.528	0.5059
7.95	6.93	8	7.56	20	22.26	0.501	0.5005
4.45	4.45	8.25	8.26	40	32.72	0.62662	0.6272
8.2	12.21	8.5	8.44	250	250	0.6631	0.6579
4.3	3.99	11	10.83	160	171.4	0.641286	0.6263
6.2	7.88	15.5	17.25	30	29.29	0.539	0.5931
4	3.32	12	13.34	80	80	0.5136	0.5829
5	5.60	22	22.56	170	174.5	0.752	0.7538
10	13.03	13.5	11.58	150	123.6	0.894	0.7842
3.5	3.48	17.85	13.34	120	86	0.929	0.7843
3.43	3.48	18.75	22.56	100	87.87	0.9067	0.7808
6	8.41	19	14.72	150	170.7	0.663	0.772
14.12	13.03	12.8	19.38	4	4.347	0.7	0.765
7.01	12.61	14	15.96	380	337.4	1.35	1.262
2.16	2.43	24.22	19.38	150	182.8	1.153	1.149
6.46	5.81	20.5	18.24	40	43.58	1.39	1.314
3.4	2.43	12	15.96	20	17.57	1.149	1.126
5.74	3.06	10	14.25	260	239	1.018	1.096
6.74	8.54	19	23.65	80	84.36	1.012	0.974
1.04	1.08	17	20.52	50	50.21	1.357	1.061
24	23.24	21.5	20.94	250	275.3	2.323	2.438
35	26.7	27	26.7	346.67	388.7	2.24	2.035
73.16	75.13	31	35.59	135	113.4	2.757	3.021
25.22	21.64	20	22.64	73.33	50.21	2.437	2.304
29	23.23	20	20.94	245	275.4	2.236	2.442
31.85	34.93	27.25	26.7			2.232	2.039

Appendix F

Table F-1: Mean nutrient concentration of influent and effluent from the vegetated and control systems for different retention times

Parameter		Retention time (days)				
		Concentration (µg/l)				
TP (mg/l)		**0.5**	**1**	**2**	**3**	**6**
	Influent	4.133	4.867	4.979	4.961	4.689
	Vegetated	2.184	1.178	0.783	0.526	0.394
	Control	2.476	1.546	1.000	0.897	0.798
SRP (mg/l)		**0.5**	**1**	**2**	**3**	**6**
	Influent	3.177	3.753	3.427	3.645	4.024
	Vegetated	1.500	8.10	0.474	0.301	0.296
	Control	2.171	1.328	0.519	429.23	0.414
NH_4-N (mg/l)		**0.5**	**1**	**2**	**3**	**6**
	Influent	6.665	11.117	7.524	10.528	10.271
	Vegetated	0.146	0.083	0.104	0.483	0.475
	Control	0.313	0.134	0.397	0.980	2.105
NO_3-N (mg/l)		**0.5**	**1**	**2**	**3**	**6**
	Influent	0.129	0.127	0.162	0.142	0.103
	vegetated	3.613	12.760	4.91	8.080	4.361
	Control	0.999	3.521	22.317	20.215	5.388
NO_2-N (mg/l)		**0.5**	**1**	**2**	**3**	**6**
	Influent	0.027	0.175	0.113	0.243	0.190
	Vegetated	0.050	0.080	0.059	0.191	0.166
	Control	24.83	20.23	1069.71	794.19	271.19

Appendix G

Table G-1: Nutrients concentration for the different sampling sites along Njoro River from October 2009 to February 2010

Parameter	Njoro bridge					Bora milk					Egerton bridge					Discharge point				
	Oct	Nov	Dec	Jan	Feb	Oct	Nov	Dec	Jan	Feb	Oct	Nov	Dec	Jan	Feb	Oct	Nov	Dec	Jan	Feb
TP mg/l	0.22	0.28	0.22	0.16	0.14	0.18	0.22	0.19	0.14	0.14	0.31	0.24	0.17	0.21	0.21	0.14	0.25	2.31	1.78	3.24
SRP mg/l	0.11	0.11	0.10	0.07	0.07	0.10	0.09	0.09	0.06	0.05	0.14	0.08	0.08	0.06	0.09	0.12	0.08	2.10	1.57	3.01
NH4 - N mg/l	0.10	0.09	0.07	0.04	0.08	0.11	0.06	0.16	0.05	0.09	0.25	0.11	0.16	0.08	0.07	0.19	0.10	5.55	4.13	8.01
NO3-N mg/l	0.42	0.77	0.69	0.56	0.46	0.50	1.10	0.59	0.68	0.99	0.30	0.82	0.57	0.90	1.27	0.54	0.99	1.25	0.73	0.73
NO2-N mg/l	0.17	0.23	0.17	0.10	0.25	0.28	0.27	0.37	0.16	0.24	0.51	0.29	0.34	0.11	0.33	0.26	0.24	0.43	0.49	0.47

Appendix H

Table H-1: Physico-chemical parameters measured in effluent from Egerton University WSPs from November 2009 to July 2010. Bolded values are means ± Standard Errors, n = 80

Months	Concentration values										
	EC (µs/cm)	TDS (mg/l)	pH	Salinity (g/l)	Temp (°C)	TSS (mg/l)	Organic (mg/l)	Inorganic (mg/l)	DO (mg/l)	BOD5 (mg/l)	Coliform (MPN/100ml)
Nov ± SE	**0.77**±0.22	**357**±292	**8.67**±0.03	**0.5**±0.00	**18.5**±1.5	**113.5**±26.5	**104.5**±24.5	**9**±2	**8.5**±0.51	**133.21**±52.03	**2109**±989
Dec ± SE	**1**±0.01	**649.5**±8.50	**8.75**±0.05	**0.5**±0.00	**18**±0.8	**89**+4	**82.5**±1.5	**6.5**±2.5	**6.61**±0.81	**125.25**±3.62	**2079**±721
Jan ± SE	**1**±0.00	**600**±0.01	**9.14**±0.04	**0.4**±0.00	**22.9**±0.00	**144**±0.56	**34**±10.49	**114**±1.56	**6.42**±0.89	**273.5**±6.89	**1600**±320
Feb ± SE	**540** ± 0.00	**0.92**±0.02	**9.20**±0.13	**0.47**±0.03	**21.7**±1.2	**86**±12.85	**68.67**±10.35	**17.33**±7.51	**8.91**±2.05	**348.33**±45.29	**2482**±1457
Mar ± SE	**0.83**±0.00	**591.33**±10.27	**8.58**±0.4	**0.4**±0.00	**21.7**±0.2	**82**±9.87	**78**±8.78	**4**±1.02	**7.80**±0.76	**400**±38.45	**2370**±210
Apr ± SE	**0.77**±0.01	**498.17**±5.02	**8.05**±0.17	**0.4**±0.00	**19.3**±0.4	**71.17**+13.73	**63.33**±12.05	**7.83**±2.12	**7.78**±1.45	**249.87**±39.27	**2249** ±793
May ± SE	**0.71**±0.02	**501.57**±6.75	**7.88** ±0.05	**0.4**±0.00	**19.9**±0.5	**59**±6.6	**54.5**±5.82	**4.5**±1.44	**4.02**+0.55	**161.29**±34.22	**1915**±668
June ± SE	**0.69**±0.03	**491**±23.29	**7.94**±0.2	**0.38**±0.05	**20.2**±0.5	**63.06**±10.86	**59.06**±8.46	**4**±2.41	**6.08**±0.93	**113.57**±10.62	**665**±231
July ± SE	**0.57**±0.01	**419.5**±6.76	**7.75**±0.22	**0.3**±0.00	**18.5**±0.5	**80.25**±13.35	**69.75**±9.86	**10.5**±4.03	**6.75**±0.29	**84.23**±27.63	**1600**±372

Appendix I

Table I – 1: Nutrient parameters measured in effluent from Egerton University WSPs from November 2009 to July 2010. Bolded values are means ± Standard Error, n = 105

Months	TP (µg/l)	SRP (µg/l)	NH4- N (µg/l)	NO3-N (µg/l)	NO2-N (µg/l)
Nov ± SE	**7636.65** ±58.71	**6695.09** ± 94.62	**12125.68** ± 243.6	**283.99** ± 197.93	**33.12** ± 1.94
Dec ± SE	**7636.65** ± 246.6	**6566.41** ± 117.33	**9907.6** ± 350.51	**213.71** ± 76.02	**53.66** ± 8.46
Jan ± SE	**7468.34** ± 230	**6782.14** ± 106.4	**2977.29** ± 456.32	**1325.29** ± 132.3	**14.53** ± 24.78
Feb ± SE	**6882.87** ± 204.68	**6512.16** ± 146.41	**2352.96** ± 175.7	**255.31** ± 11.94	**223.49** ± 44.54
Mar ± SE	**6834.24** ± 231.3	**4867.09** ± 87.56	**3545.68** ± 136.78	**154.91** ± 87.53	**82.4** ± 35.85
April ± SE	**4914.53** ± 102.18	**3866.67** ± 180.15	**8453.69** ± 1419.42	**127.85** ± 27.36	**193.59** ± 58.15
May ± SE	**4871.64** ± 146.39	**3961.33** ± 102.92	**13446.13** ±463.29	**119.25** ± 9.73	**51.85** ± 11.09
June ± SE	**4736.06** ± 213.9	**3684.8** ± 146.9	**8079.28** ±1247.52	**167.81** ± 32.0	**288.16** ± 121.58
July ± SE	**4222.9** ±294.3	**2894.4** ±132.62	**1735.62** ± 386.71	**165.66** ±17.89	**90.37** ± 20.59

Appendix J

Table J-1: Physico- chemical and microbial parameters measured in effluent from of the pilot CW from April 2010 to July 2010. Bolded values are means ± Standard Errors, n = 60

Month	EC (µs/cm)	TDS (mg/l)	pH	Salinity (g/l)	Temp (°C)	TSS (mg/l)	Organic (mg/l)	Inorganic (mg/l)	DO (mg/l)	BOD5 (mg/l)	Coliform (MPN/100ml)
April	**0.61** ± 0.03	**398.53** ± 22.39	**7.26** ± 0.08	**0.3** ± 0.02	**19.93** ± 0.67	**20.04** ± 3.36	**14.30** ± 1.55	**5.74** ± 2.21	**5.58** ± 0.81	**8.59** ± 1.40	**178** ± 80
May	**0.57** ± 0.01	**408.5** ±7.41	**7.23** ± 0.14	**0.3** ± 0.00	**20.03** ± 0.36	**16.54** ± 2.05	**13.96** ± 1.69	**2.59** ± 0.55	**5.02** ± 0.10	**10.68** ± 2.47	**79** ± 31
June	**0.60** ± 0.02	**428.46** ± 17.02	**7.07** ± 0.04	**0.33** ± 0.02	**20.12** ± 0.48	**14** ± 2.09	**11.06** ± 3.13	**2.94** ± 1.98	**6.35** ± 0.53	**5.30** ± 0.42	**72.92** ± 21
July	**0.49** ± 0.03	**369.2** ± 22.03	**7.09** ± 0.04	**0.27** ± 0.02	**18.03** ± 0.5	**9.45** ± 2.25	**8.58** ± 2.23	**0.87** ± 0.14	**7.05** ± 0.18	**12.09** ± 7.10	**35** ± 8

Appendix K: Nutrient parameters measured in effluent Pilot CW between April 2010 to July 2010. Bolded values are means ± Standard Error, n = 75

Month	TP (µg/l)	SRP (µg/l)	NH4-N (µg/l)	NO3-N (µg/l)	NO2-N (µg/l)
April	**500.13** ± 87.63	**199.18** ± 46.02	**672.36** ± 184.59	**4495.94** ± 1886.01	**292**.07 ± 120.66
May	**1085.12** ± 265.72	**739.31** ± 160.26	**356.37** ± 106.03	**11771.1** ± 2250.22	**83**.30 ± 19.64
June	**886.35** ± 237.81	**504.8** ± 181.3	**68.16** ± 16.51	**4991**.36 ± 981.91	**68.59** ± 37.93
July	**1308.33** ± 333.13	**1126.30** ± 317.77	**50.61** ± 11.37	**1050**.74 ± 75.14	**11.72** ± 3.40

Appendix L

Table L-1: Nutrient parameters measured in effluent Pilot control system from April 2010 to July 2010. Bolded values are means ± Standard Error, n=75

Month	TP (µg/l)	SRP (µg/l)	NH4- N (µg/l)	NO3-N (µg/l)	NO2 -N(µg/l)
April	**788.23** ± 67.06	**421.37** ± 77.10	**1883.30** ± 552.50	**14110.92** ± 4200.21	**559.55** ± 133.35
May	**1546.74** ± 0.7	**1400.08** ± 71.92	**124.09** ± 10.41	**3912.05** ± 390.85	**12.26** ± 7.96
June	**947.76** ± 43.43	**571.68** ± 56.97	**123.7** ± 34.28	**24272.3** ± 245.8	**1463.77** ± 37.98
July	**2477.02** ± 1.00	**2135.21** ± 36.50	**146.7** ± 0.5	**999.38** ± 0.60	**24.74** ± 0.09

Appendix M

Table M-1: Physico- chemical and microbial parameters measured in effluent from of the pilot control system from April 2010 to July 2010. Bolded values are means ± Standard Errors, n = 60

Month	EC (µs/cm)	TDS (mg/l)	pH	Salinity (g/l)	Temp (°C)	TSS (mg/l)	Organic (mg/l)	Inorganic (mg/l)	DO (mg/l)	BOD (mg/l)	Coliform (MPN/100ml)
April	**0.55** ± 0.02	**400.4**± 13.46	**7.12**± 0.05	**0.3**± 0.00	**19.6**± 0.5	**11.1**± 3.30	**7.45**± 2.15	**3.65**± 1.36	**8.55**± 0.95	**7.72**± 1.67	**76**± 24
May	**0.5**± 0.04	**379.25**± 38.75	**7.22**± 0.08	**0.3**±0.00	**17.4**± 0.6	**13**± 7.5	**7.25**± 3.25	**5.75**± 4.25	**10.48**± 0.55	**18.14**± 10.40	**1225**± 1175
June	**0.57**± 0.03	**407.5**± 0.03	**7.1**± 0.02	**0.4**± 0.00	**18.9**± 0.2	**29**± 4.5	**19**± 0.34	**10**± 0.23	**5.14**± 0.32	**10.67**± 0.05	**50**± 120
July	**0.60**± 0.00	**477**± 22.0	**7.65**± 0.22	**0.4**± 0.00	**18.7**± 0.1	**32.5**± 1.5	**22.25**± 0.25	**8.25**± 0.25	**5.75**± 0.03	**9.22**± 0.22	**57.5**± 2.5