DEVELOPMENT AND TESTING OF AN ON-SITE DOMESTIC GREY WATER TREATMENT SYSTEM USING CRUSHED BRICKS AND MAIZE COB CHARCOAL AS TREATMENT MEDIUM

BY

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DECLARATION

Declaration by the Candidate

This thesis is my original work and has not been presented for a degree in any other University. No part of this thesis may be reproduced without the prior written permission of the author and/or Moi University.

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This work is dedicated to my wife, Mrs Lucy Akinyi Otieno, my son, George Griffins Onyango and my cousin brothers and sisters for their unending support during the period of my study.

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ABSTRACT

Developing countries have low levels of access to piped water supply and sanitation. Most household in rural and poor urban areas lack water connections or wastewater disposal systems. Water is fetched from distant sources mostly by women and children. Recycling and reuse of water is an option for the partial reclamation of the value invested in the water and reducing the effluent load on the open drains that serve for sewers. The main objective of this study was to develop and test a biological household level greywater treatment system using locally available materials to a level of reusing the treated water as an alternative source for non-portable uses within communities. The specific objectives were; (i) to design and fabricate an appropriate greywater treatment system fit for treating greywater from kitchen, (ii) to run a lab-scale treatment technology assessment for the designed system to evaluate the performance of crushed bricks mixed with maize cob charcoal used in the designed system as the treatment medium and (iii) to evaluate the system run time for maximum performance. The study used greywater collected from Moi University Ngeria students' cafeteria kitchen that was passed through a test unit with anaerobic tank filled with wood charcoal and a series of five slanted filter beds stacked above each other and filled with treatment media of crushed bricks (B) and maize cob charcoal (C) both of mean diameter 10mm ±2 mixed in a C:B ratio of 1:3. A similar arrangement was also set up for a control with only crushed bricks as the treatment media. The flow through the units was by gravity at a domestic freshwater demand of 72 l/head/day. Sampling for both influent and effluent from the anaerobic chambers and filter beds were collected weekly and analysed for selected water quality parameters characterized for domestic greywater including BOD_5, NH_4, TN and TP. It was observed that effective removal of organic matter (BOD_5) in both systems began one week after start up. The anaerobic chambers on average removed 55.1% of the organic loading in the greywater by reducing the BOD_5 from 544.3mg/l in the raw greywater to 231.2mg/l. The water quality improvement for the test unit in the removal of the parameters were: BOD_5 - 544.3 to 6.1mg/l (99%), NH_4 – 1.3 to 0.49 mg/l (62%), TN – 32.7 to 3.6mg/l (89%) and TP – 6.5 to 0.6mg/l (90%). For the control unit in the removal of these parameters were: BOD_5 – 544.3 to 21.8mg/l (96%), NH_4 – 1.3 to 0.6mg/l (54%), TN – 32.7 to 3.5mg/l (89%) and TP – 6.5 to 1.0mg/l(85%). The BOD_5 concentrations recorded in the effluent from the test system was below 10mg/l with the lowest at 1.2mg/l in the 5th week for the first 16 weeks indicating run time of 16 weeks of effective operation. In the 17th week, it recorded the first high of 13.0mg/l which kept on rising to a maximum of 18.9mg/l in the 22nd week. Analysis of the results from the two units indicated a better performance of the test unit in removing the measured parameters which could be associated with the incorporation of maize cob charcoal in the crushed bricks due to the porous nature of charcoal which provided a suitable habitat for microbes that helped in clarifying greywater biologically. The test system was effective for 16 weeks when it was operating below 10mg/l recommended by WHO standards for lawn and flower watering, dust control among other non-portable uses.

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ACRONYMS

AF:	Anaerobic Filter
AH:	Anaerobic Hybrid
APR:	Anaerobic Piston Reactor
BOD_5:	Biological Oxygen Demand at day 5
BS:	British Standards
CFU:	Colony-Forming Unit
COD:	Chemical Oxygen Demand
CEPT:	Chemical Enhanced Primary Treatment
CO_2:	Carbon dioxide
CH_4:	Methane
DAF:	Dissolved Air Filter
EU:	European Union
FAO:	Food and Agriculture Organization
HLGW:	High Load Grey Water
LAVICORD:	Lake Victoria Comprehensive Research for Development
LLGW:	Low Load Grey Water
MCM:	Million Cubic Metre
MSL:	Multi-Soil Layering
NTU:	Nephelometric Turbidity Unit
N_2:	Nitrogen gas
NH_4:	Ammonium ion
NO_2:	Nitrite
NO_3:	Nitrate
OWDTS:	On-site Wastewater Differentiable Treatment System

PHE:	Public Health Engineering
PPM:	Parts Per Million
PAO:	Phosphate Accumulating Organisms
PO_4:	Phosphates
PTF:	Polyurethene-foam Trickling Filter
RBC:	Rotating Biological Contactor
TSS:	Total Suspended Solid
T-N:	Total Nitrogen
T-P:	Total Phosphorus
UASB:	Upflow Anaerobic Sludge Blanket
USA:	United States of America
USEPA:	United States Environment Protection Agency
UK:	United Kingdom
WRA:	Water Resources Authority
WHO:	World Health Organization

SYMBOLS AND NOMENCLATURE

H_0: Null hypothesis

H_a: Alternative hypothesis

m_i: Population means

T: Test statistics

N_i: Sample sizes

$\bar{Y}_i$: Sample means

S_i^2: Sample variances

a: Significant level

t: t-distribution

v: Degrees of freedom

d_e: Effective diameter

U_c: Uniformity coefficient

$\%R$: Percent removal

C_i: Influent concentration

C_e: Effluent concentration

CHAPTER 1: INTRODUCTION

1.1 General Overview

Freshwater scarcity is generally a reality in many regions around the world today, Kenya being among the many nations affected. The pressure on this finite resource has been hastened by the world's increasing population and economic developments. The increased demand and hence its scarcity therefore calls for a scrutiny of present water use strategies. The new look points out at rational and sensible use of the already available water. This can be achieved by recycling and reusing wastewater that is increasingly being generated due to rapid population growth and related developmental activities including agricultural and industrial production (Vigneswaran and Sundaravadivel, 2004).

Kenya is a water scarce country with about 47.6 million people, of which close to 43% have no access to clean and safe water (Marshall, 2011). The freshwater scarcity has been a serious issue in the country for decades, resulting from years of recurrent droughts, poor management of water supply, pollution of the available water resources and an acute increase in water demand as a result of rapid population growth. The acute water shortage in the country calls for alternative water sources to supplement the limited available sources. Reuse of wastewater could be one possible alternative if properly treated to meet quality standards for domestic water. Kenya generates a lot of wastewaters from industries and households which go into drains though the country is chronically water scarce and has one of the world's lowest water replenishment rates per capita (Kaluli et al., 2011). This means that all the generated wastewaters will always find their ways into natural water bodies which are the only water sources for low-income communities which use the water directly from the sources and causing serious pollution resulting to waterborne diseases and other ailments among these communities.

Water pollution problem due to untreated wastewater discharges is recognized globally and efforts have been made towards formation of quality standards for wastewater discharge into water bodies. Mostly, the widely applied discharge standard is the “20/30 standard” (BOD5 $\leq$ 20 mg/l and SS $\leq$ 30 mg/l) developed in the United Kingdom and later adapted and adopted by most other countries in the world (Morel and Diener , 2006). However, in water scarce regions, notably Middle East and Sub-Saharan Africa, contaminant concentrations may be high due to little water used and these discharge standards will hardly be met.

Wastewater is any water that is no longer needed, as no further benefits can be derived out of it. About 99% of wastewater is water, and only 1% is solid wastes (Vigneswaran and Sundaravadivel, 2004). Domestic wastewater is categorized into greywater and black with black water being described as wastewater originating from toilets and is grossly contaminated by faecal coliform and generally has high concentrations of organic matter. Greywater is that wastewater originating from bathrooms, laundries and kitchen and constitutes the largest flow of wastewater. According to World Health Organization, greywater represents about 61% of the total wastewater stream, (WHO, 2006). The greywater quantities produced by any household greatly varies according to the dynamics of the household and is influenced by factors such as the number of occupants, their age distribution, lifestyle characteristics, water use pattern, the cost of water and climatic conditions (Thaher et al., 2020).

Just like quantity, greywater quality in a household varies daily depending on the activities of the occupants and the source where it is generated - kitchen, laundry, bathroom, etc. Greywater from most households contain soap, shampoo, toothpaste, shaving cream, laundry detergents, body oils, dirt, and grease among other things. The most significant pollutants in greywater are soap and laundry detergents, particularly those high in sodium and phosphate. If kitchen wastewater is part of the grey water, then high organic loading and cooking oil and fats are also expected. The organics in the waste will finally decompose into ammonia, nitrates, and nitrites (WHO, 2006).

Greywater contains significant amounts of nutrients (nitrogen and phosphorus). An average volume of greywater (356 L/day) will produce approximately 45g of nitrogen and 3g of phosphate per day (WHO, 2006). Greywater from bathtubs, showers and hand washbasins is considered as the least polluted greywater source. Dishwashing and laundry detergents are the main sources of phosphorous in greywater. Kitchen greywater is the main source of nitrogen in domestic greywater, while the lowest levels are generally observed in bathroom and laundry greywater.

The history of wastewater reuse dates back to 5000 years ago in the Minoan civilization in ancient Greece where they developed an interest in the potential of wastewater reuse to overcome water shortages for irrigation purposes. Other countries like Germany, UK, China and India also reported the use of wastewater for irrigation purposes between the 16^{th} and 18^{th} centuries. Over the last three decades, many national governments have recognized the need to reuse wastewater as a means of supplementing water resources and protecting the environment (Vigneswaran and Sundaravadivel, 2004).

A variety of technologies have been used and some are being developed for treatment and recycling of greywater. Most treatment units reported in the literature are based on physical processes - filtration and disinfection, while the more current ones incorporate biological treatment as well (Friedler et al., 2005). Selection of the most appropriate technology is dependent on many factors such as the scale of operation, end use of the water, socioeconomic factors relating to cost of water and regional customs and practices (WWAP, 2017). In rural areas, with adequate land, ‘natural’ treatment systems seem to be appropriate while in urban areas, the treatment technologies selected should have a small footprint due to space constraints.

On-site wastewater treatment is a decentralized system that needs simple, reliable, low-energy consuming and low-cost technology that private homeowners with little skills for operations can manage (Oladoja, 2016). It entails passing wastewater through some kind of treatment media (bioreactor) which uses physio-chemical (filtration, adsorption) and biological (microbial degradation) processes to purify wastewater. Amongst the materials that have been used as bioreactor in on-site treatment systems are soil materials such as sand, loam, peat (Oladoja et al., 2006). The use of soil as an on-site treatment media is restricted by soil permeability. However, addition of other materials like sawdust, charcoal iron fillings can improve on the permeability as well as its ability to adsorb soluble pollutants.

Soil is an effective treatment medium for any wastewater treatment since it contains a complex biological community. One table spoonful of soil can contain over a million microscopic organisms, including bacteria, protozoa, fungi, moulds, and other creatures (Hyngstrom et al., 2011). These bacteria and other microorganisms help in treating wastewater by degrading the organics present in it. But the wastewater must pass through the soil slowly enough to provide adequate contact time with microorganisms. Due to soil’s physical characteristics and as a site for biological activities, it can treat wastewater through mechanical filtration of particles in the wastewater, removal of some chemicals and nutrients through adsorption and acting as a site for destroying pathogens.

The focus of this research was to use soil in a modified form (crushed and graded bricks) combined with charcoal produced from maize cobs under specified conditions as a treatment media in an on-site domestic wastewater treatment and recycling system. The system would treat greywater from households and convert it into a useable source that would substitute the precious drinking water in other household uses which do not require potable water quality such as toilet flushing, garden irrigation, car washing and dust control. Soil modification was necessary since in its natural state, soil is less permeable and can also impact on the turbidity of the final water (treated water). Charcoal was added to improve on the adsorption capacity of the treatment media since charcoal is very porous and provides large surface area for adsorption. It is also a good habitat for bacteria for the degradation of organics. The treatment system was designed to have a holding tank, a pre-treatment unit - a tank with charcoal filled to about halfway to provide anaerobic conditions and the bio-chemical treatment unit. The pre-treatment unit was incorporated to help reduce BOD_5 by about 50% through anaerobic degradation of the organics, and also help in settling some of the settleable organics which would clog the main treatment unit (Itayama et al., 2006). A parallel system using crushed bricks alone as the treatment media was also set up as a control and was running concurrently with the one under study. It was used as a control because other researchers have used bricks for wastewater treatment and this was used as the baseline data. The research was therefore to determine the effect of charcoal if used to amend the crushed bricks, as well as do a comparison between the two systems.

1.2 Problem Statement

Kenya is a water scarce country, and this is a limiting factor towards economic development. Therefore, there is need for water saving and water enhancement strategies. In many places throughout Kenya, low-income communities do not have a household water connection. In these communities, women and children often have to walk long distances or wait in line in order to access water which then needs to be carried home. As a result, children lose part of their time for education while women on the other side could use this time for doing other things which can raise their living standards. In such households, the water that is brought home is highly valuable because of the amount of labour invested and the cost relative to household income (Nnaji et al., 2013).

On the other hand, Kenya generates a lot of wastewaters which find itself into the natural water bodies. This is a major concern as these water bodies get polluted from the wastewater discharges leading to loss of some aquatic lives. Low-income communities living around the water bodies also collect their domestic water from the polluted sources and use it for portable purposes as raw as collected without any form of treatment. The result is the increase of water-borne diseases such as cholera which do affect these communities. An example of such environmental concerns is Lake Victoria which regularly experiences upsurge of water hyacinth and algal bloom due to wastewater discharges from the neighbouring communities as well as industries around. However, the communities around the lake still collect their household water from the lake and use it at home with no kind of treatment. This explains the rampant cholera outbreaks around the lake region.

With such problems at hand, it is essential to reduce the water demand for fresh water by substituting fresh water with alternative water resources and to optimize water use efficiency through reuse options. Among the alternatives which this research intends to address is the greywater. Greywater is a feasible resource since it is produced on daily basis by each household and when well-treated can be used to meet part of the existing demand for some non-portable uses which do not require freshwater quality. By so doing, time lost by both women and children to collect water will be reduced as part of the household water required will be generated at home and at the same time, wastewater discharges into the freshwater bodies will be reduced thereby reducing pollution of the natural water sources (WHO, 2006).

1.3 Objectives

The goal of this research was to contribute to attempts made in providing solutions to water scarcity problems by treating household grey water using locally available materials and reusing the treated water as an alternative/additional source of water for non-portable uses within the household in rural and low-income urban communities in Kenya.

1.3.1. Main objective

To develop and test a biological household level greywater treatment system using locally available materials to a level of reusing the treated water as an alternative source for non-portable uses within communities.

1.3.2. Specific objectives

1. To design and fabricate the appropriate greywater treatment system for treating greywater from kitchens.
2. To evaluate performance of the designed system when different materials are used as the treatment medium.
3. To evaluate the system run time.

1.4 Justification

This study is beneficial to the low-income rural and peri-urban communities by reducing their water bills through reusing water for toilet flushing, irrigating their lawns and gardens and dust control among other non-portable uses. Water use efficiency will be improved as the heavy labour problem of water delivery by women and children and time lost during water collection as a social problem will be addressed. The existing pressure on water resources within the communities will be reduced since there will be an alternative source of water. The environment will be well maintained, and water pollution will be reduced due to reduced discharges into water resources.

1.5 The Study Area

1.5.1 Location

The research was carried out at the PHE Laboratory at the Department of Civil and Structural Engineering, School of Engineering, Moi University Main Campus in Kenya at latitude $0^0$17’ N, longitude $35^0$17’ E and at an altitude of 2215 meters above sea level for a period of 12 months. The scope of the work included review of literature materials, material acquisition, construction of a small section within the lab, design and setting up the experiments, sample collection, analysis and report writing.

1.5.2 Climate

Moi University is in a highland equatorial climate area with an average annual rainfall of about 1124mm (Ochieng G., 2001). This rainfall is reliable with the probability of obtaining more than 750mm of rainfall in a year being about 90% to 100%. The rainfall is evenly distributed and occurs in one long season (March to September) with two distinct peaks in May and August. The dry spells fall between November and end of February. The average temperature in the county is about 18^0C during the wet season with a maximum of 26^0C during the dry season and a minimum of about 8^0C in the coolest season (Ochieng G., 2001).

CHAPTER 2: LITERATURE REVIEW

2.1 Global Water Demand

Water is a basic need for both human survival and socio-economic development. The quantity of water resources on the planet earth is much higher than the amount required by the humans living on it. However, about 97% of the resources is in the oceans and seas and is too saline for most production purposes. About two thirds of the remainder is tied up in ice caps, glaciers, permafrost, swamps, and deep aquifers and therefore not easily accessed. Of the freshwater, only 0.3% is available from rivers, lakes and reservoirs (Vigneswaran and Sundaravadivel, 2004). This is illustrated in figures 2.1 and 2.2

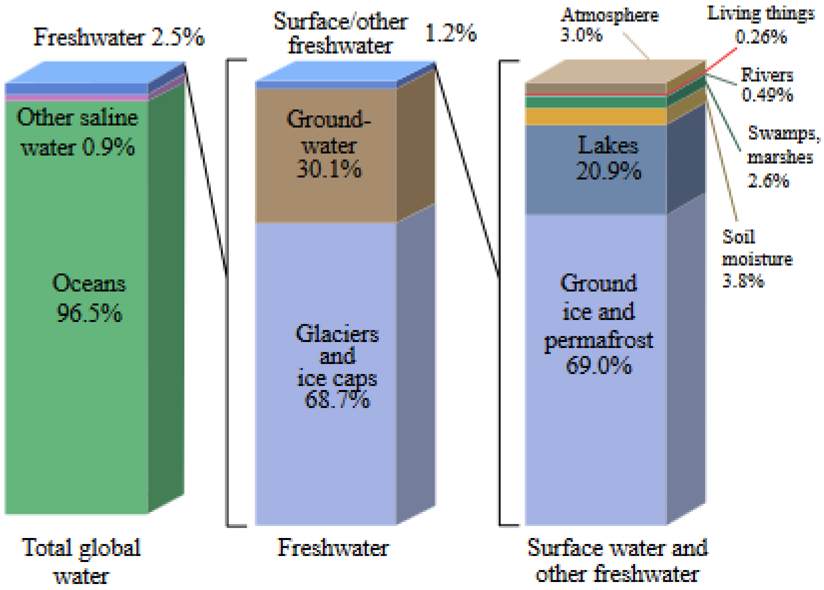

Figure 2.1: Distribution of the Earth's water (Parliman, 2016)

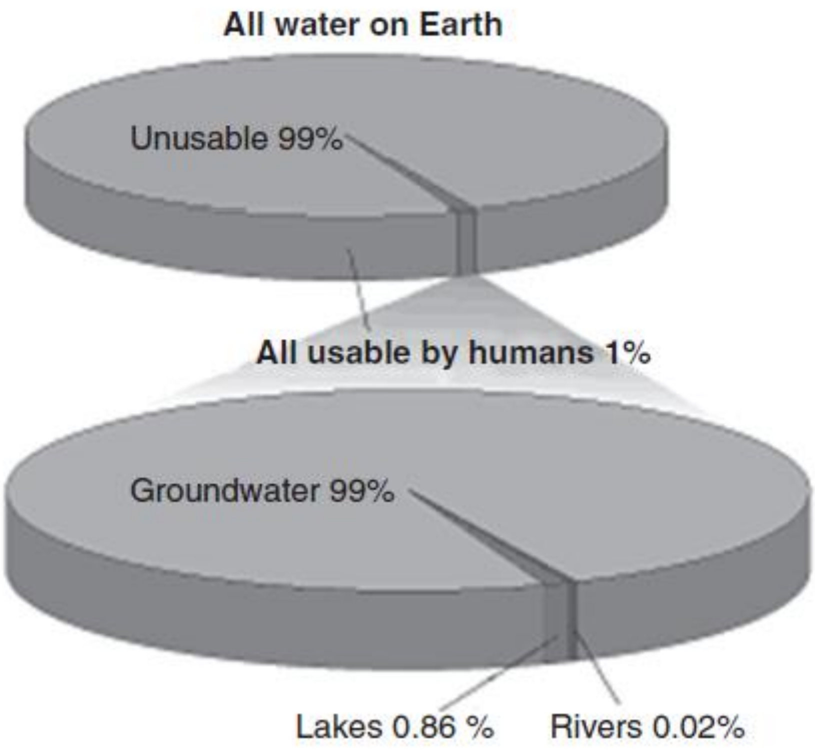

Figure 2.2: Water available on Earth for human consumption (Plessis, 2017)

Unfortunately, the freshwater available for consumption is increasingly getting contaminated (Desai et al., 2013; Kamble et al., 2017). Annually, about 119,000 km^3 of water precipitate on the land surface and 458,000Km^3 precipitate on the oceans. Of the 119,000Km^3 falling on the land surface, close to 61% (72,000 km^3) evaporates directly back into the atmosphere, leaving 44,700 km^3 flowing towards the sea (Chow, 1988). If this amount were evenly distributed, it would be approximately 9,000 m^3 per person per year which is far much more than enough for human consumption. However, much of the flow occurs in seasonal floods. It is estimated that only between 9,000 km^3 and 14,000 km^3 may be ultimately controlled. Currently, only 3,400 km^3 are withdrawn for use (Seckler, 1998) and this is not evenly distributed in time and space over the globe (Vigneswaran and Sundaravadivel, 2004). Fewer than 10 countries possess about 60% of the world's available fresh water supply and as a result, freshwater scarcity is a reality in many regions of the world today. Globally, water use increased six-fold during the twentieth century and by the year 2025 about 1.8 billion people will live in absolute water scarce conditions, i.e., with an annual water supply of less than 500 m^3 per capita, and two-thirds of the world's population will experience water stress, i.e., will have an annual water supply of less than 1700 m3 per capita. By 2030, the planet will host 8 billion people with the highest growth expected in developing countries, where 82% of the world's population already lives and experiences water scarcity (Dalahmeh, 2013).

Despite its scarce and uneven distribution nature, water resources demand for household, commercial and agricultural uses are significantly increasing around the globe due to urbanisation, population growth and increased economic activity (Friedler, 2004; Vigneswaran et al, 2004; Kamble et al., 2017; Sidibe, 2014). The ever-increasing water demand has led to scarcity even in regions that were initially perceived to be water sufficient such as Europe and Japan.

2.2 Water Demand and Supply Gap in Kenya

In the year 2010, it was noted that Kenya as a country was already water scarce with a total water demand of 3,218 million m^3 (MCM) against a deficit of 1,418 MCM giving a 44% deficit. The demand was projected to grow to 21,468 MCM against a deficit of 14,958 MCM (70% deficit) in the year 2030 (WARMA, 2013). In many places around the country, water scarcity is a limiting factor towards economic development and therefore there is need for water saving and water enhancement strategies. The water supply was estimated to be about 650 m^3/capita/year, by the year 2011 and could go down to about 350 m^3/year by the year 2020 (Kalui et al., 2011).

According to The Ministry of Water and Irrigation (2005), freshwater consumption rate in Kenya ranges from 10 litres/capita/day for low potential people without individual water connections in the rural set up to 250 litres/capita/day for high class housing with individual connections in the urban areas. The maximum consumption rates for both rural and urban set-ups are 60 and 250 litres/capita in a day.

2.3 Need for Alternative Sources of Water

The question about where extra water is to come from has prompted a scrutiny of present water use strategies. There is development of new water sources including seawater desalination, exploitation of more distant (surface water) and deeper (groundwater) sources (Friedler, 2004), building of bigger dams and reservoirs and diversion canals (Vigneswaran and Sundaravadivel, 2004) and rainwater harvesting (Nnaji et al., 2013). However, the cost of utilizing such sources is so high that the urban poor and the rural majority cannot meet. They are even much expensive compared to the conventional methods of water treatment. Besides, they have increasing negative environmental effects. For example, seawater desalination results in increased CO_2 and other pollutants emission to the atmosphere. It also causes disturbance to the adjacent marine environment (Friedler et al., 2005).

As the world's population grows, the generation of wastewater increases and for this reason, wastewater can be regarded as a reliable source of water that can reduce the increase in freshwater demand when it is suitably reused (Sidibe, 2014). Therefore, during or before coming up with new water sources, a thorough examination of the overall water consumption is necessary to promote utilisation efficiency, to enhance water saving measures and to reuse wastewater as an alternative resource (Friedler, 2004). Water should be regarded as a finite resource that has to be recycled and reused to preserve it (Desai et al., 2013). One promising option to this is the on-site wastewater recycling system which has great potential of making rational use of already available water, which if used sensibly; could provide sufficient water for all. The approach invariably points out at recycle and reuse of wastewater that is being increasingly generated due to rapid growth of population and related developmental activities, including agriculture and industrial productions (Vigneswaran and Sundaravadivel, 2004). Reports by (Devotta et al., 2007) indicate that recyclable wastewater will meet 15% of total water requirement by the year 2050.

According to Jhansi et al., (2013), water scarcity and water pollution are very crucial issues in the world of today. One way of reducing the effects of water scarcity and pollution is to advocate for water and wastewater reuse. The escalating water scarcity in the world in combination with the rapid population growth in urban areas raises concerns about proper water management practices (Mekki et al., 2015). In relation to trends in urban development, wastewater treatment and recycling deserves greater emphasis.

Kenya's Vision 2030 addresses the lack of equity in the availability and distribution of fresh and potable water for all by advocating for the conservation of water sources, rainwater harvesting, and enhancing the utilisation of ground water. The existing water policy does not include wastewater reuse (Kalui et al., 2011). The Ministry of Water and Irrigation goal to provide water in sufficient quantity and quality by 2010 was not achieved. Therefore, there is need to identify innovative ways of bridging the existing water supply gap and meet Kenya's industrial, domestic, and agricultural water needs. One such idea is the adoption of wastewater treatment and reuse.

2.4 Wastewater Recycling

Water resources are very critical for economic growth of a country and yet it is often wasted (WHO, 2006). Water conservation and wastewater reuse has of late received a lot of attention in arid and semi-arid regions across the globe. This is because, the conservative use and reuse of wastewater is key in achieving sustainability of water supply to communities in these areas and thus the socioeconomic development. Greywater availability is not dependent on season or variable like rainwater and therefore its treatment and reuse can make a continuous and a reliable water resource (Desai et al., 2013). It therefore requires smaller storage facilities than rainwater harvesting.

There is precious value in water and each drop should be accounted for especially in dry regions such as the Middle East and North Africa (Vigneswaran and Sundaravadivel, 2004). Thus, wastewater should be identified as a renewable resource rather than a waste as has been perceived. This is because it can help in increasing water availability and at the same time, curbing environmental pollution. One main aim of recycling wastewater is to substitute the precious drinking water in applications which do not require drinking water quality (Mehlhart et al., 2005; Sidibe, 2014; Desai et al., 2013). About 99% of wastewater is water and only 1% is solid waste (Vigneswaran and Sundaravadivel, 2004). To utilize this resource, there is need for collection, treatment, and reuse of all generated wastewater. In a developing urban society, the average wastewater produced usually ranges between $30m^3$ to $70m^3$/capita/year. In a city of about one million people, the wastewater generated would be enough to irrigate roughly 1500-3500 hectares of land (Jhansi et al., 2013). Greywater recycling in the urban sector may significantly reduce the general urban water consumption, leading towards a more sustainable urban water utilisation (Friedler et al., 2005).

The domestic in-house specific water demand in developing countries ranges between 60 and 150 litres/capita/day (Friedler, 2004; Friedler et al., 2005; Dalahmeh, 2013). Out of this amount about 60-70% is given out as greywater, with daily generation rates usually ranging from 30 to 120 litres/capita/day in low and middle-income countries, while most of the remainder is utilized for toilet flushing which converts to blackwater.

There are also other reports of 50-80% composition of the household wastewater as greywater (Maiga et al., 2015; Mehlhart et al., 2005; Sidibe, 2014; Desai et al., 2013). The quantities of greywater produced varies from household to household depending on the dynamics of the household including the number, age distribution, lifestyle and water-usage of the occupants, the water cost and climate. Water-usage surveys conducted in different cities of the world indicate an average wastewater generation of 586 litres/day/household. Out of this, greywater constitutes about 61% with an average flow of 356 litres/day/household (WHO, 2006).

Table 2.1: Likely constituents of greywater from various household sources (WHO, 2006)

Greywater source	**Possible contaminants**
Clothes washing	Suspended solids (dirt, lint), organic material, oil and grease, sodium, nitrates and phosphates (from detergent), increased salinity and pH, bleach
Dish washing	Organic material and suspended solids (from food), bacteria, increased salinity and pH, fat, oil and grease, detergent
Bathtub and shower	Bacteria, hair, organic material and suspended solids (skin, particles, lint), oil and grease, soap and detergent residue
Sinks, including kitchen	Bacteria, organic matter and suspended solids (food particles), fat, oil and grease, soap and detergent residue

Recycling of greywater for toilet flushing can significantly bring down the in-house net water consumption by 40-60 litres/capita/day, giving a reduction of 10-20% of the urban water demand. This is a significant saving especially under water scarce situations (Nnaji et al., 2013; Friedler, 2004). Further reduction of demand can be achieved by reusing greywater for garden irrigation which is a great water user in some semi-arid regions such as Australia, California, and Israel (Friedler, 2004; Friedler et al., 2005). A reduction of household water consumption of 30-60% is possible if greywater is reused (Sidibe, 2014). In Israel for example, it is estimated that by the year 2023, with a moderate penetration ratio of greywater recycling systems of 20-30%

(percentage of houses having greywater reuse units installed), greywater reuse in the urban sector could save 30-55 million cubic meters per year in the proportions of 25-45 and 5-10 million cubic metres per year in toilet flushing and garden irrigation respectively (Friedler et al., 2005). This attributes to about 5% of the future total urban water demand in the country and equals the capacity of a medium size seawater desalination plant. As a matter of fact, over 50% of the water demand for domestic activities can be met by treating and reusing greywater in activities including toilet flushing, gardening and car washing (Desai et al., 2013). Greywater recycling and reuse is therefore a sufficient and cost-effective way of managing water scarcity.

The greatest challenge in the water and sanitation sector to overcome in the next two decades is the implementation of low-cost sewage treatment systems that will at the same time permit selective reuse of treated effluents for agricultural and industrial purposes (Jhansi et al., 2013). It is therefore very important that sanitation systems be of high levels of hygienic standards to prevent the spread of disease.

The main goals for reuse of wastewater as stated by (Vigneswaran and Sundaravadivel, 2004), include:

- Opportunities to supplement limited primary water sources.
- Prevention of excessive diversion of water from alternative uses, including the natural environment.
- Possibilities to manage in-situ water sources.
- Minimization of infrastructure costs, including total treatment and discharge costs.
- Reduction and elimination of discharges of wastewater (treated or untreated) into receiving environment.

- Nutrient and water resource recovery for reuse in agricultural production

Wastewater treatment in urban areas has received less attention in comparison to water supply. Scarcity of water in addition to the rising populations of cities and towns have taken a toll order on health and environment. The pollution of lakes, rivers, and domestic water bodies with sewage has reached hazardous levels. The current urban wastewater management systems are linear treatment systems aimed at disposal (Jhansi et al., 2013). The traditional system should be changed into a sustainable, closed-loop urban wastewater management system on the basis of conservation of water and nutrient resources which instead of being used in agricultural food production, contaminate freshwater resources. One way of attaining this sustainability is by decentralizing the wastewater management system. The decentralized system has several smaller units serving individual houses, clusters of houses, or small communities. Greywater can therefore be treated and reused separately from the hygienically more dangerous black water. This approach results in treatment and recycling of wastewater and the by-products of the technology (energy, sludge, and mineralized nutrients) in the exact location of the system. The concept of domestic greywater reuse has been studied lately mostly in the EU, Japan USA, and Australia. However, this concept is still quite new, full-scale systems are not common, and even less have been tested for a long time.

Although there is a dropping per capita freshwater availability in Kenya, there is increasing dominance of wastewater in the water balance and this makes wastewater a very important source of irrigation water for urban agriculture. Wastewater if reused in its raw form can significantly become a non-point source of pollution. This can lead to serious health risks when people are exposed to the polluted wastewater. If adequately treated, wastewater recycling in Kenya can greatly reduce scarcity of irrigation and potable water in the country. Kenya has urban population of more than 100,000 people in towns such as Nairobi, Mombasa, Nakuru, Kitale, and Eldoret among others (Kalui et al., 2011). This population has a great potential of providing enough wastewater for industrial reuse if properly treated. The treated wastewater can be used in boilers, toilet flushing, laundry, air conditioning, cooling and processing, power generation and heavy construction.

2.5 Uses for Recycled Wastewater

Most Large-scale reuse schemes are in Israel, South Africa, and arid areas of USA, where alternative sources of water are limited (Jhansi et al., 2013). In these places, wastewater is mostly used for toilet flushing as compared to other uses within the urban environment. The reuse of wastewater for toilet flushing can reduce water demand within a dwelling by up to 30% (Jefferson *et al*, 2004; Nnaji *et al.,* 2013). For instance, the current specific water demand for residential toilet flushing in Israel averages 55 litres/capita/day (Friedler, 2004). This value is projected to go down to about 40 litres/capita/day in the future given the level of greywater reuse in the country. The other uses of wastewater may include ornamental uses in fountains and waterfalls, landscaping, lawn irrigation and car washing (WHO, 2006), industrial reuse such as fire protection, boiler feed water, and concrete production, though some special treatment may be required for heat transfer units such as boilers and power generation facilities (Jefferson *et a.l*, 2004; Nnaji *et al*., 2013; Desai et al., 2013; Sidibe, 2014), non-portable purposes, indirect portable purposes (occurs when wastewater unintentionally mixes with the receiving bodies which are sources of portable water or through planned scheme) and direct portable purpose in the household (Vigneswaran and Sundaravadivel, 2004). The last three uses require that greywater be separated from blackwater and the treatment of the greywater must strictly meet certain stipulated quality criteria. Wastewater reuse can also be a method of water resources management. For instance, topping up of depleted aquifers through injection of highly treated wastewater thereby restoring aquifer yield.

2.6 Wastewater Reuse in Kenya

In Kenya, poor urban communities use wastewater for irrigation without any form of treatment. A study done in 2006 and 2007 indicated that only 50% of the wastewater generated in Nairobi finally reaches the treatment facilities while the rest is used in irrigating over 750 ha of land while still raw (Kalui et al., 2011). This happens in areas including Kahawa, Soweto, Kibira, among other low-income communities living in Nairobi and its environs. About 75% of the crops grown in these areas, especially the vegetables, is sold while the rest is consumed by the farmers. The wastewater is used by the farmers because it not only provides soil moisture but also provides the nutrients necessary for plant growth. The use of raw wastewater for irrigation poses health risks to the consumers of these farm produce.

2.7 Guidelines for Wastewater Reuse

Most countries do not have standards or guidelines for reuse of wastewater and therefore use guidelines from World Health Organisation (WHO) or United States Environmental Protection Agency (USEPA) for any decision to reuse wastewater. In Kenya, no person is permitted to use wastewater for irrigation purposes unless such water complies with the quality guidelines in the eighth schedule of the Environmental Management and Coordination, (Water Quality) Regulations (NEMA, 2006) stipulated in table 2.2.

Table 2.2: Microbiological quality guidelines for wastewater use in irrigation in Kenya (NEMA, 2006)

Reuse conditions	Exposed group	Intestinal nematodes (MPN/L)*	Coliforms (MPN/100 ml)
Unrestricted irrigation (crops likely to be eaten uncooked, sports fields, public parks)	Workers, consumers, public	<1	<1000**
Restricted irrigation (cereal crops, industrial crops, fodder crops, pasture and trees***	Workers	<1	No standard recommended

The quality standards for sources of domestic water as provided by the (NEMA, 2006) guidelines indicate a maximum allowable value of 10Mg/L of Nitrate-NO_3, 0.5Mg/L of Ammonia-NH_3 and 3Mg/L of Nitrite-NO_2. For effluent discharge into the environment, the NEMA standards allows 30Mg/l of Biochemical Oxygen Demand (BOD_5) and 2 Guideline Values for both Total Nitrogen and Total Phosphorus.

The available guidelines requirement for each application of wastewater are geo-specific but normally contain criteria based on organic, solids and microbiological content of the water. Some of the most restrictive standards criteria for instant South Korea require a BOD_5 of less than 8 mg/l and turbidity below 2 NTU for food crops irrigation and a non-detectable level of either total or faecal coliforms (Jeong, 2016). However other countries do not have very stringent guidelines and can allow higher concentrations of the different parameters or do not include them at all (Jefferson et a.l, 2004).

Almost all the guidelines and standards for reuse of wastewater are mainly for irrigation purposes because irrigation is the highest consumer of water in any country and therefore given the priority in reuse of wastewater (Vigneswaran and Sundaravadivel, 2004). Table 2.3 shows some guidelines for given parameters in treated greywater depending on the type of use.

Table 2.3: Permitted limit for greywater reuse according to the use type (WHO, 2006)

Parameter	**Permitted limit for a given purpose / type of reuse**		
	Irrigation of ornamentals, fruit trees, and fodder crops	Irrigation of vegetables likely to be eaten uncooked	Toilet flushing
Biological Oxygen Demand, BOD_5 (mg/l)	≤ 240	≤ 20	≤ 10
Total suspended solids TSS (mg/l)	≤ 140	≤ 20	≤ 10
Fecal coliforms (Cfu/100 mL)	≤ 1000	≤ 200	≤ 10

According to (Kalui et al., 2011), wastewater reuse should meet the following guidelines:

- Wastewater treatment system should reduce pathogen concentrations to a point that it meets the WHO (1989) guidelines.
- Crop restrictions must be specified to prevent direct exposure to those consuming uncooked crops as well as defining application methods that reduce the contact of wastewater with edible crops,
- Control of human exposure is needed for workers, crop-handlers, and final consumers.

The Food and Agriculture Organization (FAO) standards needed for restricted irrigation, where the food produced does not come into contact with contaminated soils are BOD_5 <25 mg/l, TSS <35 mg/l and faecal coliform <200/100 ml for 80% of the samples. For unrestricted irrigation, where different irrigation methods could be used, making the crop vulnerable to contamination, the standards are higher; BOD_5 <10 mg/l, TSS <10 mg/l and faecal coliform <5/100 ml for 80% of the samples (Kalui et al., 2011). Standards recommended for urban reuse of greywater in activities including toilet flushing, vehicle washing, landscape irrigation and fire protection are 10Mg/L of BOD_5. (Nnaji et al., 2013).

2.8 Types of Wastewaters and their Quality

Wastewater is any used water with dissolved or suspended solids, discharged from homes, commercial establishments, farms, and industries. Domestic wastewater can be categorized into blackwater and greywater (WHO, 2006), of which blackwater is from toilets and kitchens and consequently have adverse faecal coliform contamination as well as high concentrations of organic matter. Greywater on the other hand is generated from bathrooms and laundries which contributes the largest portion of domestic wastewater. Contrary to the WHO (2006), Lo´pez-Zavala (2007) stated that household wastewater can be differentiated by The Onsite Wastewater Differentiable Treatment System (OWDTS) into three categories. These include reduced-volume blackwater, higher-load greywater (HLGW) and lower load greywater (LLGW). The classification of greywater into high load and low load was also reported by Mehlhart et al. (2005) and Sidibe (2014) and this depends on the greywater source. For instance, Jefferson et al. (2004) grouped kitchen and cloth washing greywater as High load since they are heavily polluted and greywater from bath tabs, showers, and hand basins as low load.

Greywater generally means any untreated household wastewater with the exception of wastewater from toilet (Mehlhart et al., 2005; Morel et al., 2006). The constituents of greywater include wastewater from bathtubs, showers, hand basins, laundry tubs, floor wastes and washing machines (WHO, 2006). However, there are conflicting information in literature on the classification of kitchen wastewater. Some literature materials (Jefferson et a.l, 2004; Devotta et al., 2007; Mehlhart et al., 2005; Desai et al., 2013; Morel et al., 2006) group it under greywater while others group it under blackwater. However, (Morel and Diener , 2006) classified kitchen water as greywater but clearly indicated that greywater from kitchen requires special attention. If kept for sometimes, greywater degrades and after reaching septic conditions, form sludge which can either settle or float depending on the nature of the organics present and the climatic conditions.

Just like the quantities, the greywater quality also varies between and even within households daily in relation to the activities of the inhabitants (WHO, 2006; Morel et al., 2006). The greywater source will also determine its quality. The major constituents of households' greywater include soap, shampoo, toothpaste, shaving cream, laundry detergents, hair, lint, body oils, dirt, grease, fats, and urine if infant clothing's are included. Greywater from hand washing, bath tab and showers contribute to about 50-60% of the total greywater and is considered to be the least polluted. Some faecal contamination including associated bacteria and viruses may also be present in it because of body washing. Cloth washing contributes about 25-35% of the total greywater. The quality of cloth washing greywater varies from wash water to rinse water to second rinse water. If kitchen wastewater forms part of the greywater, it contributes to about 10% of the total greywater volume. It is composed of food waste, oils and fats including other chemical pollutants such as detergents and cleaning agents which are alkaline in nature and contain various chemicals (Devotta et al., 2007). These result into either chemical or bacteriological contamination of the greywater. The key chemical contaminants are the nitrates originating from the biodegradation of the organics (food waste) in the greywater (Mehlhart et al., 2005) and phosphorus from the detergents. Other contaminants include colour, turbidity, organic materials, bacteria (aerobic and anaerobic) and viruses.

Although greywater may be perceived to be less polluted than blackwater, it may be highly polluted with faecal coliforms of about 104-108 CFU/100 ml, and significant concentrations of detergents and salts (Friedler et al., 2005). Therefore, greywater may still be risky for human health and pose environmental and aesthetic effects, particularly in warm climates with higher ambient temperatures which accelerate organic matter degradation and enhance growth of pathogens. The organic matter load in greywater in terms of COD ranges between 13 and 8000 Mg/l. These organic loadings originate from detergent, food, dirt and skin residues and are highly degradable under both aerobic, and anaerobic conditions. The nitrogen load is reported to be ranging between 0.6 and 74 Mg/l. The phosphorus concentrations is however, guided by the regulations put by a country on the use of phosphorus-containing detergents. Phosphorus concentrations can vary between 4-14 Mg /L in non-phosphorus detergent greywater and 6-23 Mg/L in areas where phosphorus-containing detergents are still in use (Dalahmeh, 2013).

A study by Friedler (2004), to characterize both quantity and quality of greywater streams discharged from domestic appliances indicated low levels of heavy metals which were below the detection limits. The study however identified the washing machine, dishwasher and kitchen sink as the major pollutant generators with COD concentrations in the range of 1,300 mg/l, BOD_5 up to 700 mg/l, phosphate up to 500 mg/l, and chlorides and sodium in the range of 600-700 mg/l. Of all the appliances, the study revealed that kitchen sink was signalled out as the major contributor of greywater (with 25 litres/capita/day), while the dishwasher as the least producer (with only 5 litres/capita/day). Based on these results on the daily greywater discharge and on the domestic daily water demand for toilet flushing, (Friedler et al., 2005) recommended treatment and reuse of light greywater, i.e., greywater originating from the bath, shower and washbasin.

Characterization of greywater from a shower from a household in Japan indicated that the greywater had an average organic load of 374.4 mg COD/L (Lo´pez-Zavala, 2007), with approximately 65% in particulate and 35% in soluble forms. Nitrogen load was about 11.6 mg T-N/L from which 88% was O-N, 6% NH4 and 6% NO_3. Other parameters characterized are as in table 2.4.

Table 2.4: Average characteristics of shower greywater from a house located in Sapporo City, Japan (Lo´pez-Zavala, 2007)

Anions	Mg/L	Cations	mg/L	Other parameters	
F	0.79	Na	21.38	COD (mg/L)	374.43
Cl	27.89	NH_4	0.74	XCOD (mg/L)	245.16
NO_2	0.00	K	3.53	SCOD (mg/L)	129.27
NO_3	0.69	Mg	0.00	T-N (mg/L)	11.59
PO_4	0.00	Ca	0.00	O-N (mg/L)	10.16
SO_4	17.00			EC (mS/cm)	237.46
				pH	6.52

In comparing the findings with the primary effluent from the municipal wastewater treatment plant, it was noted that greywater from shower contained 5% more soluble organic matter and more Organic Nitrogen than primary effluent from municipal wastewater treatment plants (Lo´pez-Zavala, 2007).

Another study of characterization of low load greywater from shower, bath tab, and hand basin indicated similar biodegradable contents as demonstrated by BOD_5 concentrations of about 146Mg/l, 129Mg/l and 155Mg/l respectively (Jefferson et a.l, 2004). However, there were many variations in terms of COD concentrations in the same sources indicated by 420Mg/l, 367Mg/l and 587Mg/l respectively.

Table 2.5 is an illustration of a homogeneous greywater from kitchen, laundry, shower, bath tab, and hand basin as specified by World Health Organization.

Table 2.5: Typical composition of greywater (WHO, 2006))

Parameter	Unit	Range
Suspended solids	mg/L	45–330
Turbidity	NTU	22–200
BOD_5	mg/L	90–290
Nitrite	mg/L	< 0.1–0.8
Ammonia	mg/L	< 0.1–25.4
Total Kjeldahl nitrogen	mg/L	2.1–31.5
Total phosphorus	mg/L	0.6–27.3
Sulphate	mg/L	7.9–110
pH	-	6.6–8.7
Conductivity	mS/cm	325–1140
Sodium	mg/L	29–230

While a homogeneous greywater from kitchen, laundry, shower, bath tab, and hand basin would be the most ideal for this research, the available greywater at the time of the study was from kitchen at the Ngeria Halls student cafeteria which was mostly for washing utensils.

2.9 Types of Wastewater Treatment Systems

The use of high-technology wastewater treatment systems such as centralized sewage plants (convectional systems) involve large initial cost of investment as well as operating and maintenance costs, and therefore for economic reasons, they do not apply in wastewater treatment for small communities as well as rural set up, especially in developing countries. Natural decentralized wastewater treatment systems have been gaining importance as the effective and low-cost alternative for rural area wastewater treatment. These systems serve approximately 25% of the population in the United States, (Chen et al., 2009).

A variety of treatment processes have been proposed in literature (Friedler et al., 2005). However, since the on-site greywater reuse is still a new practice, only a few off-theshelf systems are commercially available, and even less have been tested on full scale for long time periods. Most of these treatment systems in the literature are basically of physical processes (filtration and disinfection), while the biological processes are incorporated in the most current systems. In the rural set-ups with land availability, natural treatment systems are reported to be appropriate while in urban areas which require the highest water saving potential, the treatment technologies to be employed should have a small footprint due to space constraints. A wide range of technologies have been employed or are being developed for the treatment of greywater for reuse (Jefferson et a.l, 2004). These developments have also incorporated the use of natural treatment systems as well as basic coarse filtration, chemical processes, physical and physiochemical processes and biological processes. The choice of an appropriate technology will always depend on several factors of consideration such as the scale of operation, end use of the water, cost of water as well as cost of operation. Consequently, the nature of the wastewater, particularly the organic strength will influence selection of appropriate processes towards biological systems. Commonly used systems are membrane bioreactors, sequence batch reactors and biologically aerated filters which all produce high effluent standards. There are reports of common use of aerobic biological treatment method which uses a rotating biological contactor (RBC) (Nnaji et al., 2013). It is also reported that biological systems can effectively reduce the BOD_5 concentrations of greywater to below 10 mg/l (Jefferson et a.l, 2004).

A key objective aimed at expanding the coverage of wastewater treatment should be the application of appropriate wastewater treatment technologies that are effective, simple to operate, and low cost (initial investment, operation and maintenance). Appropriate technologies are also friendlier to the environment since they utilize less energy and thus contribute positively to the efforts made towards mitigating the effects of climate change. In relation to modern design, appropriate technologies are less of environmental nuisance than conventional systems, for instance they produce less amounts of sludge and their odour problems can be more effectively controlled.

Some of the appropriate wastewater treatment technologies (Jhansi et al., 2013) include:

- Preliminary Treatment by Rotating Micro Screens.
- Vortex Grit Chambers.
- Lagoons Treatment (Anaerobic, Facultative and Polishing), including recent developments in improving lagoon performance.
- Anaerobic Treatment processes of various types, mainly, Anaerobic Lagoons, Up flow Anaerobic Sludge Blanket (UASB) Reactors, Anaerobic Filters and Anaerobic Piston Reactor (APR).
- Physicochemical processes of various types such as Chemically Enhanced Primary Treatment (CEPT) and Constructed Wetlands.
- Stabilization Reservoirs for wastewater reuse and other purposes.
- Overland Flow.
- Infiltration-Percolation.
- Septic Tanks; and
- Submarine and Large Rivers Outfalls.

Some of the above processes can be combined to come up with one major set up. Combination with some other simple processes like Sand Filtration and Dissolved Air Floatation (DAF), which are not considered appropriate processes is also possible. One such combination is the treatment of wastewater for reuse in irrigation, based on pre-treatment by one of the above-mentioned appropriate technologies followed by a stabilization reservoir.

All the systems should permit the reuse of treated wastewater in order to possess a cyclic, sustainable system. The treated wastewaters have essential plant nutrients (nitrogen, phosphorus, and potassium) as well as trace nutrients. Phosphorus is specifically an important nutrient to reuse since the phosphorus in chemical fertilizer comes from limited fossil sources.

Having mentioned some of the appropriate technologies necessary for sustainable wastewater treatment for reuse, it is necessary to discuss a few of them from which this research borrowed ideas since they share similar functionalities. These include:

a) Lagoons/wetlands,
b) Anaerobic digesters
c) Soil Aquifer Treatment (SAT technologies) / Soil Biotechnology Treatment

2.9.1 Lagoons and wetlands

Wetland systems utilize natural processes (biological, chemical, physical, and solar) acting together in wastewater purification thus achieving wastewater treatment objective. In such systems, a series of shallow ponds act as stabilization lagoons, while water weeds and algae consume nutrients in the wastewater. In addition, water hyacinth or duckweed act to accumulate heavy metals. Various forms of bacteria (aerobic and anaerobic), breakdown the biodegradable organics in the wastewater thus improving on the biological oxygen demand. In developing countries, wetland treatment systems provide a comparative advantage over conventional treatment systems since the degree of self-sufficiency, ecological balance, and economic viability is greater, (Jhansi et al., 2013). The system provides a total resource recovery. However, a lagoon system is only considered a low-cost technology if enough of non-arable land is available for its set up. This is unfortunately not the case in big cities. In rural areas, much of the land is used for agricultural purposes. To use wetlands, you must also consider the environmental conditions prevailing hence climate is also a key factor. Other disadvantages that may make this system unsustainable to use for irrigation and other purposes in some locations include clogging with sprinkler and drip irrigation systems-particularly with oxidation pond effluent, biological growth (slime) in the sprinkler head, emitter orifice, or supply line causes plugging due to heavy concentrations of algae and suspended solids.

2.9.2 Anaerobic Digesters

In this system, anaerobic bacteria degrade organic materials in the absence of oxygen and produce methane, hydrogen sulphide and carbon dioxide as the by product. The methane gas is usable as an energy source (biogas) in households. During the digestion by the anaerobic bacteria, there is a reduction of total bio-solids volume of up to 50%-80% and the resulting waste sludge which is biologically very stable can serve as rich humus for agriculture, (Jhansi et al., 2013). This system has been applied in Colombia, Brazil, and India to substitute the more costly activated sludge processes or reduce the land requirement for wetland systems. One important advantage of the anaerobic treatment technology is that its application can either be on a very small scale or very large scale making it sustainable option for small communities.

2.9.3 Soil Aquifer Treatment

This is a geo-purification system whereby pre-treated wastewater is allowed to flow through a soil system and artificially recharges the aquifers and is then later withdrawn for use. By flowing through unsaturated soil layers, the wastewater undergoes purification before mixing with the natural groundwater. In areas with water scarcity problems, the treated wastewater becomes resourceful for improving groundwater resources. An example is The Gaza Coastal Aquifer Management Program which applies the system to improve the groundwater in terms of both quantity and quality (Jhansi et al., 2013). Having reduced the quantities of nitrogen in the pre-treatment systems, the soil system has a potential of further reducing the concentration of nitrates in the aquifer. This is done through nitrification followed by de-nitrification process which convert the nitrogen load in the wastewater to elemental nitrogen gas (Kamble et al., 2017). In areas such as the Middle East and parts of Southern Africa which are water scarce, wastewater has become a useful resource of which, after adequate treatment, becomes an alternative for groundwater recharge, agriculture, and urban applications.

Soil Aquifer Treatment systems are cheap, efficient for pathogen removal and are easily operated technically, with the only operational cost associated with it being the pumping cost for pumping the water from the recovery wells. The systems typically clear the wastewater of all BOD, TSS, and pathogenic organisms and tend to achieve standards that would generally allow unrestricted irrigation. The greatest advantage of this system is that it eliminates the pipe-to-pipe connection associated with reusing treated wastewater from a treatment plant.

The required pre-treatment for Soil Aquifer Treatment System vary depending on intended use of the recharged groundwater, wastewater source, recharge methods, and location. Some pre-treatments may only require primary treatment or treatment in a stabilization pond. However, pre-treatment methods should be avoided in case they leave high concentrations of algae in the recharge water. Algae can lead to severe clogging of the soil in the infiltration basin. As much as the system can give better quality of treated water compared to the source wastewater, the quality will still be lower than that of the original groundwater. Therefore, the system should be designed and managed properly to avoid intrusion into the groundwater and only use part of the aquifer. The gap between infiltration basins and wells or drains should be made large, usually at least 45 to 106 m to allow for adequate soil-aquifer treatment, (Jhansi et al., 2013).

2.10 Multi-soil-layering system for wastewater treatment

Multi-Soil-Layering system is a technology utilizing natural soil to facilitate wastewater treatment (Pattnaika et al., 2007; Chen et al., 2009). It is a biphasic layered system using locally available materials such as soil, iron particles, jute or sawdust, charcoal, and zeolite or alternative materials (Chen et al., 2009; Pattnaika et al., 2007; Attanandana et al., 2000; Latrach et al., 2015; Gulhane M., Yadav P., 2014). The materials are systematically arranged as in figure2.1 below. The system has been tested in Japan, Thailand China and USA to treat domestic and restaurant wastewater as well as polluted river water (Latrach et al., 2015). It effectively reduces the levels of inorganic pollutants like nitrate, ammonium, and phosphate, as well as organics as indicated by high COD and BOD. The mixing of iron particles improves the phosphate fixing capacity of the soil by dissolving in the anaerobic zone and then transferred to the aerobic zone to provide coating to the zeolite for fixing of phosphate (Attanandana et al., 2000). The inclusion of the homogeneous coarse particles of zeolite reduces the clogging of the system. A charcoal layer enhances the degradation of BOD_5. Nitrification process is also enhanced by the charcoal, zeolite and the aeration process while denitrification process is enhanced by the addition of carbon source (sawdust or kenaf plus corncob) and iron filings. The multi-soil-layering system comprises of two layers which are aerobic and anaerobic. The aerobic layers consist of zeolite or Perlite and this alternates with anaerobic layers of soil mixture blocks. Its efficiency in wastewater purification is dependent on the relative effectiveness of aerobic and anaerobic layers. In the aerobic layer, nitrification, oxidation, and precipitation of mobile ferrous iron to high-surface area ferric oxide is enhanced resulting to improved phosphorus sorption (Pattnaika et al., 2007). In the anaerobic layer of the soil mixture block, nitrate is converted to nitrous oxide and nitrogen gas through denitrification and ferric iron is reduced to the more mobile ferrous iron, which can easily move out of the anaerobic layer. Maintenance of a Multi-Soil-Layering system is simple and its effective life is estimated to be longer than 10 years even though an appropriate amount and timing of aeration is necessary. The efficiency for this system is high with 95% of BOD, 75% of total nitrate and 80% of total phosphate removal rate (Attanandana et al., 2000; Pattnaika et al., 2007).

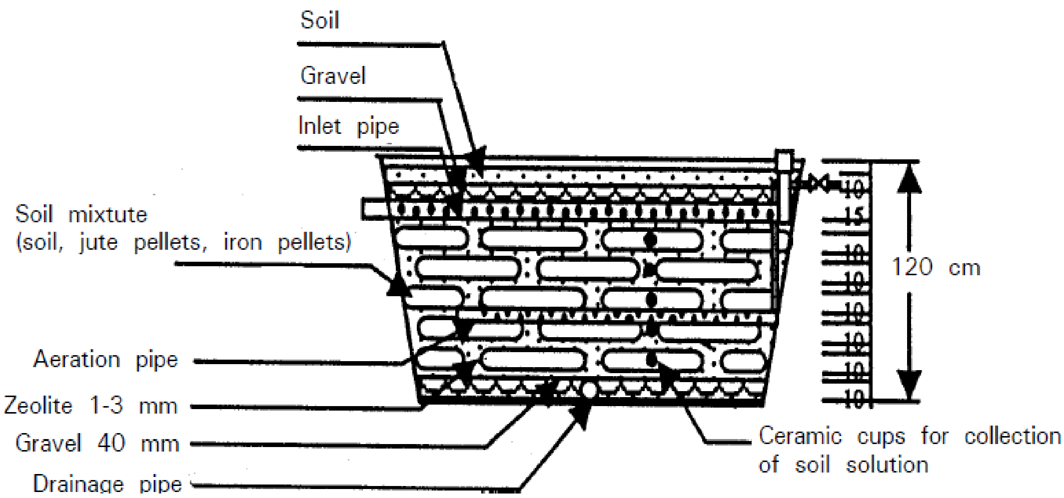

Figure 2.3: Components of the MSL system (Pattnaika et al., 2007)

2.11 Materials Used in Onsite Greywater Treatment Systems as Bio Filters

The term bio-filtration refers to the biodegradation of pollutants by microorganisms fixed to a porous media (Sidibe, 2014). Filter medium acts as a basis for physical filtration, physio-chemical adsorption, and microbial growth. The use of filters of various carrier materials with different filter pore sizes is a common approach to greywater treatment (Dalahmeh, 2013). There are reports on the use of macro pore filters, including simple strainer and mesh filters, nylon sock filters, gravel filters, sand filters, charcoal filters and tree back filters (Dalahmeh, 2013), natural soil and pebble filters (Lo´pez-Zavala, 2007; Oladoja et al., 2006; (Kamble et al., 2017), polyurethane-foam trickling filter (Elmitwalli et al., 2003), kanuma soil and crushed baked mud bricks (Ushijima et al., 2013), wood chips, wheat straw, kaldnes plastic and a mixture of compost and wood chips (Sidibe, 2014). The macro pore size ranges from 1-5 mm for coarse pores and 0.075-1 mm for fine pores. The coarse macro pore filters can only filter out particles and hair but cannot significantly remove suspended pollutants. This results in low reduction of turbidity, chemical properties and organic loading in the raw wastewater, which may lead to biological growth in recycling system. Sand filters, which are within the fine macrospore range, are the most applied filters for on-site treatment of greywater. However, clogging is one major problem associated with the sand filters.

The use of micro and ultra-membrane filters, with pore size 0.1 µm for greywater treatment, have also been reported (Dalahmeh, 2013). These micro and ultra-filters can treat greywater to standards which are not restricted for reuse. However, a serious problem with these filters is due to fouling and energy consumption, making membrane technology uneconomical and not feasible in small-scale applications in low- and middle-income countries.

2.12 Use of Soil in Wastewater Treatment

Soil is an effective treatment mechanism for wastewater because of its physical characteristic and as site for biological activities (Hyngstrom et al., 2011). The ability of soil to treat contaminants in wastewater depends on several soil factors (Loomis, 1999):

a. Soil physical properties - soil texture and structure

b. Soil chemical properties - the amount of soil particle surface area and their chemical properties,
c. Soil biological properties - the presence of soil microbes that can utilize or degrade incoming pollutants. It is actually stated by the University of Minnesota in their Onsite Sewage Treatment Program report that a tablespoon full of soil can have over a million microscopic organisms such as bacteria, protozoa, fungi, moulds and other microorganisms (University of Minnesota, 2011) , and;
d. Environmental conditions present in the soil.

Apart from these soil factors, chemical composition of the wastewater is one single most important non-soil factor governing the extent of wastewater treatment in soils (Loomis, 1999). Soil is naturally a hostile environment for pathogens present in wastewater as a result of its temperature, moisture and soil predators. As wastewater flows through the soil, it is treated through a range of processes such as physical filtration of solid particles (Dawes, 2006), chemical and nutrients removal and destruction of pathogens (Hyngstrom et al., 2011). Some of the microorganisms in the soil feed on the organic matter present in the wastewater. It is very important however to allow the wastewater to pass through the soil slowly enough to give it the required contact time with the microorganisms for adequate treatment. For example, for proper treatment of septic tank effluent, it is very vital to provide at least 1 metre of soil (aerated or un-aerated) for adequate contact time to be achieved. Effluent loading is also a factor during the treatment. The soil system acts on the wastewater physically, chemically and biologically.

The soil microorganisms attach themselves to soil particles using microbial slimes and use the oxygen and water that are present in the soil pores. The microorganisms require some basic conditions to live and grow: a place to live, food to eat, water, oxygen to breathe, suitable temperatures, and time to grow (University of Minnesota, 2011). By passing wastewater through soil, it means you have improved the living conditions for the microorganisms by supplying them with more food in the form of organic waste and water. This may lead to multiplication of their numbers until saturation after which some will have to die. Aerobic bacteria are more effective in braking down organic materials than anaerobic ones. Therefore, it is advisable that there is little clogging of the soil system for efficient treatment to occur (Hyngstrom et al., 2011).

There are some soil conditions which must be met for significant wastewater treatment to take place. Soil texture and structure are very important physical properties due to their influence on soil hydraulic characteristics, such as infiltration rates and permeability (Lo´pez-Zavala, 2007; Loomis, 1999). If the soil is not well structured and of good texture, it requires some special modifications to effectively treat wastewater (University of Minnesota, 2011). This is where incorporation of the soil with other materials such as charcoal, saw dust, gravels, and pebbles, among others comes in.

Soil microbiological processes are sensitive to soil environmental conditions like temperature, oxygen levels, and moisture status (Loomis, 1999). Cold temperatures lower biological performance and treatment efficiency. Likewise, as oxygen levels reduce the efficiency and types of treatment processes are changed from aerobic to anaerobic significantly.

2.12.1 Mechanisms of wastewater treatment in the soil

Soil particles have electrical charges and the soil system itself is a habitat for biological community (University of Minnesota, 2011). All these are needed for the treatment of wastewater. As wastewater passes through the soil system, soil particles provide the surface areas that it must come into contact with as it moves. This contact enables treatment of the wastewater through mechanical filtration of the larger contaminants and adsorption (attachment or binding) of others. Clayey soils are best suited for wastewater treatment since it has finer particles and pore spaces which provide more contact surface area for wastewater (Dawes, 2006). However, it clogs very fast. Soil particles are negatively charged and therefore, they can attract and bind with the positively charged pollutants. There are some minerals in the soil which can also bind with some pollutants in the wastewater and make them immobile (Hyngstrom et al., 2011). The biological community in the soil including bacteria, fungi, actinomycetes, and protozoa also feed on the organic material in the wastewater. Aerobic bacteria act optimally in aerated soil because they require oxygen. These aerobic microbial activity in the soil system mainly occurs in the first 30 cm from the soil surface (Lo´pez-Zavala, 2007). Under no oxygen conditions in the saturated areas, anaerobic bacteria act on the wastewater, but with insufficient treatment. Treatment and removal of bacteria and total suspended solids in the wastewater have been found to be taking place in the first one foot of most aerated soil trench treatment system.

2.12.1.1. Biomat / biofilm formation

As wastewater flows through a soil treatment system such as a trench, it moves through the soil media vertically to the biomat (biofilm) where anaerobic treatment takes place (University of Minnesota, 2011). Biofilm / biomat is a layer made of anaerobic bacteria that secrete a sticky substance and stick themselves to the soil particles, or any other available surfaces. The biofilm first forms along the trench bottom where ponding of the wastewater starts. Further formation of the biofilm takes place along the soil-media contact surfaces on the trench's sidewalls. A fully developed biomat layer is about one inch thick. Generally, the wastewater flow slower through the biomat than it flows through natural soil system. This allows unsaturated conditions to occur in the soil beneath the treatment trench. This condition prolongs the travel time of effluent through the soil, ensuring that it has sufficient contact time with the soil particle surfaces and the microorganisms. An efficiently working gravity-fed system should have wastewater ponded in the distribution media while the soil a few inches away from and below the distribution media should be unsaturated. If soil is unsaturated, it has pores containing both air and water. Therefore, aerobic bacteria living in these pores can effectively treat the wastewater as it travels through the soil system. Water movement is reduced in the unsaturated soil conditions and therefore the movement is aided by capillary action.

2.12.1.2 Organic load and Pathogen removal

Greywater provides a suitable substrate for bacteria in the biofilms in terms of carbon and nutrient balance (Dalahmeh, 2013). This facilitates biological mineralisation of the entrapped organic matter and oxidation of NH_4^+ leading to achievement of improved nitrification. Large particles in wastewater, including bacteria are always filtered out by soil system, while other particles are adsorbed or stick on the soil surfaces due to their opposite charges (Hyngstrom et al., 2011). Normally bacteria present in wastewater are typically large and coagulate with other bacteria or attached to solid organics in the wastewater. This makes them easily filtered out together with the suspended solids (University of Minnesota, 2011). Common disease-causing microorganisms or pathogens in septic tank effluent include helminths (septic worms), protozoa, bacteria, and viruses (Loomis, 1999). Generally, the mobility of these organisms is mostly dependent on their sizes relative to soil particles and soil pore space. Helminths are about the size of sand particles, protozoa the size of silt particles, bacteria the size of fine silt and coarse clay, and viruses the size of very fine clay. Because of their relative size, both helminths and protozoa are physically filtered out by soil, limiting their movement through soil pore spaces. Mechanical filtration and entrapment are the main treatment mechanisms which lead to final die-off of these larger septic microbes by processes such as predation by soil microbes or death due to unfavourable soil environmental factors. Viruses on the other hand are much smaller than bacteria and cannot be filtered out. However, some are positively charged making them to be easily attracted and held by the soil particles (Loomis, 1999; Hyngstrom et al., 2011). Some soil fungi produce antibiotics naturally that kill some pathogens while others are preyed on by soil predators (University of Minnesota, 2011). The survival of septic bacteria and viruses under aerobic soil conditions is poor since they do not compete well with natural soil microbes. However, if the soil system environment changes to anaerobic conditions, then the survival would shift in favour of the septic-borne anaerobes. Lower soil temperatures also favour septic bacteria and virus survival, because native soil microbe activity (predation) is low. Acid soils promote more rapid die off of most species of septic bacteria, yet encourages viral persistence, perhaps due to increased adsorption under acid conditions.

Sandy soil has limited negative charges and its main method of viral removal is by attachment to the microbial slimes laid down by soil bacteria. Studies have shown that high levels of virus removal occurs after 6 weeks of operation of sandy soil systems (greater than 85%) of less than 2-inches of sand particles at varying hydraulic loading rates (University of Minnesota, 2011). It therefore implies that it takes at least 6 weeks for the microbial slimes to fully grow in sandy systems.

2.12.1.3 Nitrogen removal

Nitrogen is of great concern because it can contaminate drinking water. As nitrogen moves through septic conditions, it undergoes several transformations. Wastewater has both organic nitrogen and ammonium ions (NH_4^+) (University of Minnesota, 2011; Loomis, 1999). The organic nitrogen in the wastewater is transformed into inorganic form (NH_4^+) in anaerobic conditions through mineralisation (Loomis, 1999) and the NH_4^+ is further transformed into NO_3^- in aerobic conditions (Dawes, 2006). The most abundant form of nitrogen entering the soil system from septic tank is in the form of ammonium (NH_4^+) ion (Hyngstrom et al., 2011). Some of this is used by bacteria, some adsorbed by soil particles while the rest converted to nitrate (NO_3) in the aerated regions within the soil system. The transportation and nature of nitrogen in a soil treatment system relies on the forms entering and the biological changes taking place as illustrated in figure 2.2.

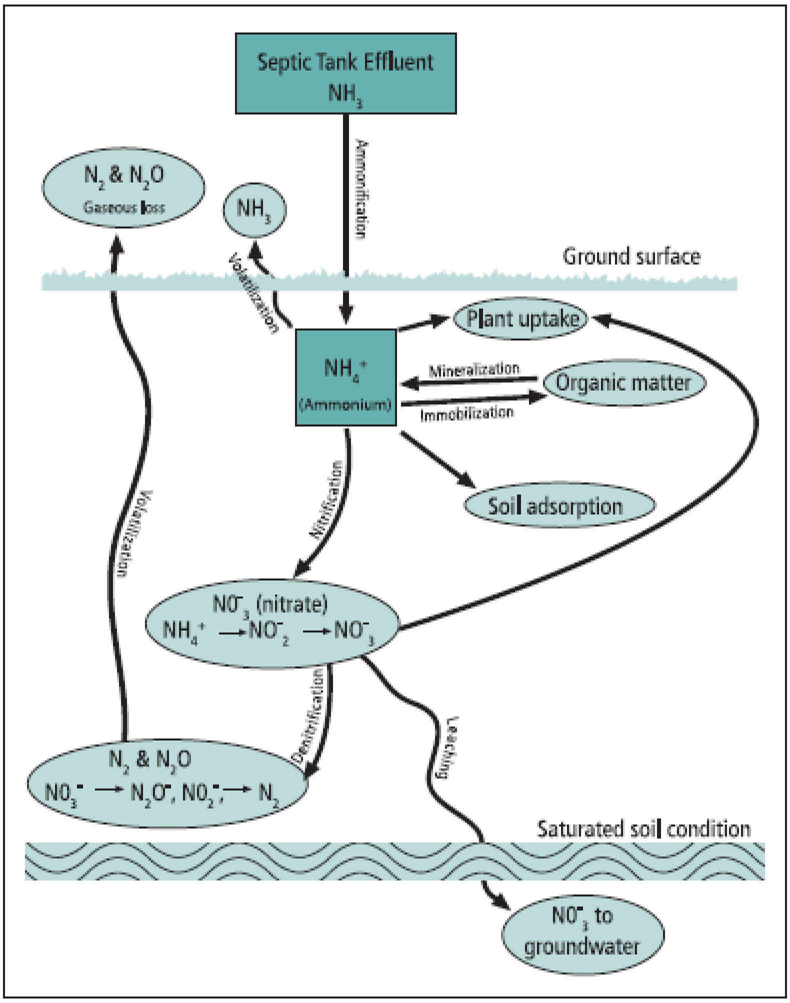

Figure 2.4: The Nitrogen Cycle and Soil Treatment (University of Minnesota, 2011)

All the above nitrogen transformations are micro-biologically catalysed and need desirable temperatures, a usable source of carbon (organic matter) for energy, and suitable alkalinity.

Nitrates (NO_3^-) are formed through nitrification process. This is an aerobic reaction and therefore it occurs in the presence of oxygen in the soil regime. Denitrification on the other hand occurs in the absence of oxygen in the soil environment below an on-site system. It helps in reducing the concentration levels of nitrates in the wastewaters. The biological denitrification (the reduction of NO_3^--N to nitrogen gases) and perhaps the most significant nitrogen removal mechanism in soil environments is done by microbes (Loomis, 1999). The end products of this process (N_2 and N_20 gases) are harmless to public health and the environment. There are four conditions which must be met in order for denitrification to effectively take place (Wu et al, 2010):

- Oxidation of NH_4^+-N to NO_3^--N (nitrification);
- Subsequent anaerobic conditions;
- Presence of denitrifying bacteria; and
- An adequate carbon (energy) source for the denitrifying bacteria present in the anaerobic zone.

The nitrogen removal process is normally facilitated by mound systems which technically reduce concentrations of nitrogen by 32 to 70% (University of Minnesota, 2011). However, in a properly designed and functioning system, the absence of reduced conditions immediately following the nitrification process and the lack of a suitable carbon source generally limit denitrification mechanisms in soil systems (Loomis, 1999). These systems normally contain efficient nitrification steps in their drain field components but lack the necessary mechanisms for denitrification to occur. There should therefore be additional steps in such systems in order to ensure that proper conditions exist for both nitrification and denitrification processes. These steps should be in discrete watertight conditions, designed to enable appropriate conditions for a particular treatment process to happen. All these steps should take place in sequence so that an initial process compliments and enhances the next subsequent treatment step. This type of scenario does not typically happen at a quantitative level in a conventional septic tank/soil absorption system.

Nitrate is soluble in water and the nitrate ions (NO_3^-) are negatively charge and cannot not be attached to soil surfaces and therefore are very mobile with water (Hyngstrom et al., 2011; Dawes, 2006).

Nitrates treatment occurs to some extent by the following mechanisms.

- **Uptake by plants:** If soil treatment areas are kept near the surface, some of the nitrate will be taken up by surface vegetation during the growing season. This is because nitrate in one of the nutrients required by plants for growth.
- **Denitrification:** If the ammonium (NH_4^+) is nitrified to nitrate (NO_3^-) in the presence of oxygen and then later moves to a saturated zone which lacks oxygen, the nitrate is transformed to nitrogen gas (N_2) and is lost to the atmosphere. During this process, there must be a source of carbon food such as dead plant materials or other organic matter (Hyngstrom et al., 2011).

The danger of nitrates in drinking water arises when it is present at high levels and may cause illnesses in infants and other vulnerable persons (Hyngstrom et al., 2011).

2.12.1.4 Phosphorus removal

Phosphorus is present in wastewater as phosphate (PO_4) and originates from some detergents (Hyngstrom et al., 2011). It is also of concern since it leads to eutrophication of water bodies receiving the wastewater with phosphate nutrients. Phosphorus is the most limiting nutrient for plant growth. Therefore, any small addition into water body brings about a great increase in plant growth, algal blooms, and heavy aquatic vegetation emergent (Loomis, 1999). Apart from making surface water bodies unpleasant for recreation, they also pose threat to the health of fish and other aquatic creatures.

Phosphate ions are negatively charged and are capable of being strongly bound to hydrous oxides of iron, aluminium, and manganese and carbonate surfaces on soil particles through a process known as adsorption, after which they are held on the soil surfaces (Loomis, 1999; Dawes, 2006; Hyngstrom et al., 2011). This process is also called chemical precipitation of phosphate (Rybicki., 1997). This process is dependent on the soil PH. Once the adsorbing surfaces are filled up, the newly added phosphorus must move deeper into the soil to find empty surfaces (University of Minnesota, 2011). Soils with higher clay content have high adsorption capacity since clay soils have bigger surface area therefore more of the adsorbed pollutants can attach themselves than soils with high sand content (Loomis, 1999). It therefore means that there is less phosphorus movement in finer-textured soils.

Modification of the soil using some of the filter materials mentioned earlier (charcoal, iron particles and saw dust) can improve the soil adsorption property in wastewater treatment, especially in the removal of nutrients and heavy metals pollutants in the wastewater (Foereid, 2015; Hollister et al., 2013). For instance, charcoal, mainly owing to its high Nitrogen sorption capacity, can be used as an effective soil amendment to reduce N losses from soils (Gai et al., 2014). However, some studies showed the opposite - a limited or no ability of charcoal to adsorb NO_3-N.

2.12.1.4.1 Biological removal of phosphates

Phosphate removal in wastewater treatment systems can also be achieved biologically. The principal advantages of biological phosphorous removal are reduced chemical costs and less sludge production as compared to chemical precipitation.

In the biological removal of phosphorous, the phosphorous in the influent wastewater is incorporated into cell biomass, which is subsequently removed from the process as a result of sludge wasting. The reactor configuration provides the Phosphate Accumulating Organisms (PAO) with a competitive advantage over other bacteria. So PAO are encouraged to grow and consume phosphorous. The reactor configuration should incorporate an anaerobic tank and aerobic reactor.

Anaerobic zone: Under anaerobic conditions, PAO assimilate fermentation products (volatile fatty acids) into storage products within the cells with the concomitant release of phosphorous from stored polyphosphates into the wastewater (Rybicki., 1997).

Aerobic zone: Energy is produced by the oxidation of storage products and polyphosphate storage within the cell increases. The soluble orthophosphate is removed from the wastewater and incorporated into polyphosphates within the bacterial cell and the new biomass with high polyphosphate storage accounts for phosphorous removal. As a portion of the biomass is wasted, the stored phosphorous is removed from the bio-treatment reactor for ultimate disposal with the waste sludge.

There are however certain conditions to be met in order to achieve this process of phosphorus removal. These conditions include Provision of a high level of simple carbon substrates in the anaerobic zone to act as a source of energy and minimized addition of electron acceptors (nitrate, oxygen) to the anaerobic zone.

2.13 Use of Charcoal in Wastewater Treatment

It has been established that charcoal has large specific surface area, high porosity, low density, and high organic content, (Dalahmeh, 2013). Charcoal also has a large capacity for adsorbing heavy metals and organic compounds. These properties indicate that charcoal has high capacity for greywater treatment, with lower risk of clogging (Shephard and Austine, 1992). Charcoal is a carbon-rich solid by-product of the pyrolysis of biomass under the complete exclusion of oxygen at temperatures below 700°C and can be derived from a large range of low-cost biomass sources, including manure, organic wastes, bioenergy crops (e.g. grasses and willows), and crop residues including maize cobs (Li J.-h., et al., 2014; Gai et al., 2014; Clough et al., 2013). At very high temperatures, the quality of the charcoal in terms of ash content will be poor since ashing temperature begins at 800^{0}C. The most direct explanation for the effect of charcoal on nutrient retention is that it acts as an adsorbent (Foereid, 2015). Previous research on the effectiveness of charcoal to reduce the level of pollutants such as Phosphate-phosphorus, Nitrogen and other organic pollutants in wastewater showed a promising outcome. Charcoal has also proven to have strong affinity for a number of heavy metal ions and therefore has great potential in removing metal contaminants from wastewater (Moges et al., 2015).

2.13.1 Mechanisms of wastewater treatment in charcoal filters

Charcoal is highly porous and therefore has a large specific surface area. Due to the large specific surface area, adsorption is generally the prevalent treatment mechanism in the initial stages of charcoal filter operation, (Dalahmeh, 2013). The charcoal surface also has hydrophobic sites consisting of carbon layers, hydrophilic functional groups, phenols and carbonyls. Such functional groups influence the charge, the acidity of the surface and the hydrophobic properties of charcoal making it capable of adsorbing a wide range of various types of pollutants such as NH_4^+ and PO_4, as well as organic matter and bacteria (Hollister et al., 2013; Foereid., 2015; Gai et al., 2014). The adsorbed organic matter provides food substrate for the bacteria inhabiting the charcoal surface. This facilitates biological mineralisation of the sorbed organic matter and oxidation of NH_4+ leading to achievement of relatively high BOD_5 and COD reduction and improved nitrification. However, the charcoal surfaces are finite resources and their adsorption capacity get exhausted with time as greywater pollutants continuously get in contact with the surfaces. Furthermore, the large specific surface area of the charcoal makes it a good habitat for a large number of biofilm bacteria and protecting them from being preyed on by the predators in the treatment system. This can lead to a scenario whereby you get more pathogen in the treated water than in the raw wastewater, especially in the later stages of filter life (Foereid., 2015). This can be attributed to overgrowth of bacteria on the large specific surface of the charcoal. Comparing the performance of natural soil and that of charcoal the sorption capacity of charcoal is high and is estimated to exceed that of soil by a factor of 10-100 (Moges et al., 2015). However, very limited research has been carried out to study the ability of charcoal to remove organic matter and nutrients, particularly ammonium and phosphate from wastewater. Moreover, a number of research on charcoal adsorption capacity is based on batch experiments.

The charcoal performance can be enhanced through activation. After initial pyrolysis, charcoal is activated by gasification with oxidising gases such as CO_2, steam or air, or phosphoric acids (Dalahmeh, 2013; Li J.-h., et al., 2014) or by addition of zinc salts (Nnaji et al., 2013). Activation results in increasing porosity and specific surface area.

2.14 Findings of Past Related Research

Experiences of greywater treatment by use of soil treatment systems have not been widely reported. However, the use of other filter media including limited use of soil and soil product materials have been reported by several researchers in the United States (Dawes, 2006), Israel (Friedler et al., 2005), India (Kamble et al., 2017), Sweden (Dalahmeh, 2013) and Japan (Ushijima et al., 2013).

A lab scale evaluation of biodegradation efficiency of Lower Load Greywater pollutants using a Controlled Soil Natural Treatment System was conducted by (Lo´pez-Zavala, 2007). The removal efficiency for organic load was about 90% for the highest infiltration rate used and 98% for the lowest infiltration. Regarding the nitrogen removal, the study established that concentrations of Kjeldahl nitrogen (K-N) in the treated effluents were of the order of 3 mg/l, down from 11.6mg/l irrespective of the change in the infiltration rates.

Experiment by Friedler et al (2005) on an On-site greywater treatment and reuse in multi-storey building in Technion campus, Israel for treating light greywater indicated a good performance of the system with the overall removal efficiency ranging from 64% of dissolved COD to 98% of turbidity. It also produced very low effluent Total BOD_5 of 2.3 mg/l and turbidity of 0.6 NTU. COD removal was much lower than Total BOD removal (96%), implying that the greywater may contain slowly non-biodegradable organics. The system successfully removed 58%, 87%, 96% and 72% of the Total Phosphate, Total Kjeldahl Nitrogen, ammonia and organic nitrogen, respectively.

Another study was conducted on the treatment of domestic sewage at low temperature in a two-step anaerobic system followed by an aerobic step (Elmitwalli et al., 2003).

The two anaerobic systems comprised of an anaerobic filter (AF) and an anaerobic hybrid (AH) while the aerobic system was made of polyurethane-foam trickling filter (PTF). The two anaerobic systems were operated at a temperature of 13^0C while the aerobic system was operated at an ambient temperature ranging from 15 to 18^0C. The finding of this research was that the overall total COD removal in the entire system was 85 %. The nutrient (Nitrogen and Phosphate) removal was limited with a phosphate removal of 25%, ammonia removal of 60% while there was increase of NO_3 from 0 Mg/l in the raw wastewater to 13.6Mg/l in the effluent from the aerobic system. Effluent from the two anaerobic systems had 0Mg/l of NO_3-N and NO_2-N while the effluent from the aerobic system had 13.6Mg/l of NO_3-N and 9.6Mg/l of NO_2-N. This is a show of nitrification process in the aerobic component of the system resulting to generation of NO_2 and NO_3.

An evaluation of the performance of a Soil Biotechnology in treating domestic wastewater in Bengaluru was conducted by (Kamble et al., 2017) in India, using a treatment media - formulated from soil with primary minerals of suitable size and composition. The findings indicated a good performance of the system with an average BOD_5 reduction of 93%, the average output NO_3-N of 23 Mg/L against the standard used of 45Mg/L and a low output of NH_4-N which met a standard of 10Mg/L. However, it was established that the nitrate level in the output water was often higher than in the input water. This was associated with the fact that the ammonium and nitrogen in the organic compounds were being converted into nitrates through nitrification, but the follow up reaction (de-nitrification) where the nitrate was to be converted to nitrogen gas did not take place sufficiently. This resulted to an overall increase in the nitrate level in the output water. The explanation offered for the scenario by the service provider of the bioreactor was that, there was insufficient organic load (COD or BOD) in the input wastewater. This organic load is necessary for the de-nitrification reaction to take place since it is a source of carbon (energy) for denitrification process.

The performance of pine tree bark, charcoal and sand filters were studied (Dalahmeh, 2013), as low-cost alternatives for on-site greywater treatment for reuse as irrigation water. The study results indicated that the organic composition and hydraulic properties of bark, together with its swelling potential, enhanced the ability of this material to effectively remove organic matter, suspended solids and pathogenic indicators from greywater compared as compared to charcoal and sand. The large specific surface area of the charcoal enhanced adsorption and the capacity for biological mineralisation of organic matter and removal of phosphorus. Charcoal was the most efficient material in removing nitrogen and phosphorus. The lower effectiveness of charcoal in removing suspended solids, pathogenic indicators and tracer microorganisms, compared with the bark, was mainly due to the charcoal having more uniform macrospores. The small specific surface of the sand and the low porosity did not support effective removal of organic matter.

The study by Ushijima et al., (2013), on the use of Slanted Soil System for greywater treatment was quite closely related to this research in terms of design and set up. In the study, kanuma soil and crushed baked mud bricks were used in a set-up of four chambers stacked above each other as shown in the (Plate 2.1)). The experiment was to compare the performance of kanuma soils and the crushed baked mud bricks. The bricks were prepared by crushing baked mud brick and sieving into 1-4 mm and 4-11 mm. The study also factored in the performance of different brick size ranges as well as a mixture of the two size ranges. Based on the research findings, the slanted soil system presented a high removal rate of both Particle COD (94-97%) and BOD_5 (88-89%), while removal rate of Dissolved COD (58-68%) was comparatively low. Sufficient pathogen removals were performed only by fine soil chamber, while most part of pathogen passed through coarse soil chamber. Disadvantage of fine soil was identified as clogging in a shorter period. However, the result of the combination of coarse and fine soil extended this time period.

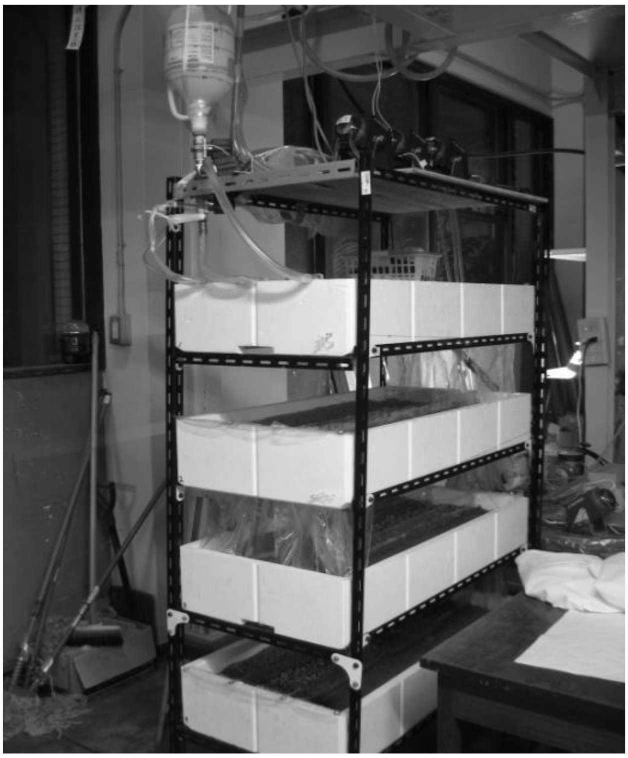

Plate 2.1: Experimental set-up by Ushijima et al., (2013)

A two-step on-site greywater treatment system to study the feasibility of sand and activated charcoal in treating greywater for developing countries was conducted by Nnaji et al., (2013). The set-up had a filtration unit made of sand as the filter media in the first step and then activated carbon in the second step as the adsorption unit. The research findings indicated that the first three weeks of running the treatment system, the BOD_5 removal was about an average of 15% while afterwards, a significant removal of as much as 85% was observed for the system. This trend was then to the increased performance of sand filtration due to the increase in the microfilm in the sand bioreactor. A very small reduction of BOD_5 was observed for the adsorption system.

The same trends as in the BOD_5 removal was also observed for the COD reduction. Despite the shaky performance of the system at the beginning of the experiment, the system approached a steady state with a performance of 85.68% BOD_5 removal, 57.09% COD removal, 70.74% TSS removal and 99.99% faecal coliform removal.

The performance of bark, biochar and activated charcoal when used as a treatment media in treating greywater was also studied (Sidibe., 2014). The reports were that all the filters were effective in the reduction of COD, especially the activated charcoal filters which had an average of 96% reduction. The effectiveness of the bark and bio-char filters in COD reduction improved during the experiment. It was further reported that in view of the diversity of common pathogens found in greywater, it can be difficult to find a single filter material that is effective in the reduction of all those pathogens.

The study done by Oladoja et al (2006) to determine the best combination ratio of stone pebbles to soil-clay to produce optimum water purification indicated that the pebbles: soil-clay ratio of 1:3 gave the highest percentage reductions of the COD (95:98%). This was an indication that fortification of soil with other materials can give good result as a way of improving the soil characteristics required for wastewater purification.

2.15 Design Procedure for On-Site Greywater Treatment and Recycling System

To design a greywater treatment and reuse system, it is important to consider the quantity of greywater to be treated and the intended reuse applications as the key factors. Therefore, design of greywater collection, treatment and reuse system must start with the quantification of greywater generated and determination of flow rate (Devotta et al., 2007).

Greywater quantification can be done through:

- **Water meter** at the outlet of the drain connecting bathrooms, kitchen and cloth washing place (laundry) or through water consumption.
- **Bucket method** where greywater is collected in a bucket of known volume at the outlet of bathroom, laundry, or kitchen.
- **Indirect method** where the quantity of water consumed can also be used to quantify greywater. This is based on the fact that the greywater quantity is averagely 60% of the total water consumption.

In this research, the indirect method was used to determine the average quantity of greywater per capita per day generated in Kenya. The quantity was again used to determine the flow rate.

The main components of a common greywater treatment system are anaerobic unit (sedimentation, septic or settling unit) and aerobic (biological filter media or bioreactors), (Devotta et al., 2007). The biological systems are reported to be the most efficient systems for treating greywater if combined with a physical treatment processes (Mehlhart et al., 2005) and can reduce the BOD_5 of the greywater to below 10 mg/l thus giving a better effluent quality than systems having only physical processes. The septic tank functions as both a quiescent zone where solids settle out of suspension, and as an anaerobic digester (Loomis., 1999). It initializes the treatment system through anaerobic degradation of some organic material and settling of solid materials thus reducing total suspended solids (TSS). TSS and organic material removal is very important because it prevents excessive clogging of the filter surface. In warmer climates, septic tanks perform far much better in terms of BOD_5 and suspended solids removal than temperate climates (Dawes, 2006). This is since the removal of suspended solids in temperate climates is due to sedimentation only while in tropical climates, the kinetics of BOD_5 is enhanced by the relatively higher temperatures resulting to mineralisation of the volatile fractions in addition to sedimentation. However, the capability of separating solids in a septic tank is maximized in the colder periods because of less gas generation and particulate re-suspension (Loomis., 1999) . Passive anaerobic treatment of septic tank wastewater for all the parameters is variable with the removal of approximately 40-60% BOD_5, 50-70% Suspended Solids 10-20% Total-Nitrogen and Less than 30% Total phosphate (Dawes, 2006). The aerobic systems provide an environment for the breakdown of pollutants in the wastewater. In the aerobic system, a substantial amount of BOD_5 and suspended solids that are not removed by simple sedimentation in a septic tank can be removed. An additional process in the aerobic system is the nitrification of ammonia in the wastewater and the significant reduction of pathogenic organisms.

2.16. Key Design Parameters

2.16.1 Residence Time / hydraulic contact time

The longer the time pollutants take in unsaturated soil, the greater the opportunity for their removal (University of Minnesota, 2011). The time that the wastewater takes to flow through a soil system for treatment is called the residence / hydraulic contact time. A way of ensuring adequate residence time so is by feeding the soil system with very low flow rates. Other methods of enhancing the contact is through the use of long, narrow, and shallow soil systems. Shallow trenches enhance good oxygen exchange with the atmosphere so that the aerobic soil bacteria can give good treatment of wastewater. Installation of water flow-restricting fixtures (baffles) in the system can also increase hydraulic contact time.

2.17 Statistical Analysis

The two-sample *t*-test is used to determine if two population means are equal. A common application is to test if a new process or treatment is superior to a current process or treatment (Snedecor et al., 1989). In some applications, you may want to adopt a new process or treatment only if it exceeds the current treatment by some threshold. In this case, we can state the null hypothesis in the form that the difference between the two populations' means is equal to some constant $m_1 - m_2 = d_0$ where the constant is the desired threshold.

The two-sample *t*-test for unpaired data is defined by:

Null hypothesis, $H_0 : m_1 - m_2 = 0$

Alternative hypothesis, $H_a: m_1 - m_2 \neq 0$

Test Statistic: $$T = \frac{\bar{Y}_1 - \bar{Y}_2}{\sqrt{S_1^2/N_1 + S_2^2/N_1}}$$ Equation 2.1

Where N_1 and N_2 are the sample sizes, $\bar{Y}_1$and $\bar{Y}_2$ are the sample means, and S_1^2 and S_2^2 are the sample variances.

If equal variances are assumed, then the formula reduces to:

$$T = \frac{\bar{Y}_1 - \bar{Y}_2}{S_p\sqrt{1/N_1 + 1/N_1}}$$ Equation 2.2

Whereby $$S_P^2 = \frac{(N_1 - 1)S_1^2 + (N_2 - 1)S_2^2}{N_1 + N_2 - 2}$$ Equation 2.3

The most used significance level is $\alpha = 0.05$. For a two-sided test, the computation is given by; $1 - \alpha/2$, or $1 - 0.05/2 = 0.975$. If the absolute value of the test statistic is greater than the critical value (0.975), then the null hypothesis is rejected.

Critical Region: Reject the null hypothesis that the two means are equal if $/T/ > t_{1-\alpha/2,v}$

Where $t_{1-\alpha/2,\, v}$ is the critical value of the t distribution with ν degrees of freedom where

$$\nu = N_1 + N_2 - 2$$

CHAPTER 3: MATERIALS AND METHODS

3.1 General Information

This chapter illustrates the processes and materials involved in this study. The first phase involved reviewing of literary materials, survey, and selection of a suitable site for setting up the pilot project within Moi University, materials and equipment acquisition, material preparation, design and fabrication of the biological filter units, doing initial runs and reviewing preliminary results. The second phase entailed sampling, laboratory testing and analysis and lasted for six months.

3.2 General Layout of the Pilot Plant

The pilot plant was situated within The Professor L. Huisman Lab at Moi University's School of Engineering. This site was selected due to the availability of and nearness to greywater which was to be sourced from the Ngeria Halls student cafeteria, convenience for lab test and analysis of the water samples and the lab was isolated and had a properly ventilated space required for wastewater treatment since wastewaters produce bad odour due to biological degradation.

Because the set-up needed protection from the public based on the nature of the equipment used and also to protect the public from the foul smell from the decomposing wastewater, a small section of the lab was isolated by constructing a room and fitted with ventilation fan for sacking out the foul air.

The pilot system was set up in a way that allowed for gravity flow of the wastewater through the filter beds. A series of five trays filled with the treatment media were stacked in a vertical rack above each other for water to flow from one tray to the next below it. Two of such systems were set-up. A quantitative flow pump was incorporated to lift the raw wastewater from the holding tank into the first chambers of treatment (anaerobic tanks).

It is always desirable to reduce the cost of setting up pilot plants by utilizing locally available and relevant materials. Based on the concern, the materials used in this research were sourced from around Moi University, while some of which couldn't be found were obtained from Eldoret town. The materials obtained within the university environs were the treatment media material (bricks and maize cob) while the construction materials including plastic trays were bought from Eldoret town.

3.3 Preparation of the Treatment Media (Bio Filter Material)

The two materials used for preparation of the treatment media were building clay bricks burnt from soil material and charcoal burnt from maize cob at a controlled temperature in a furnace. Clay bricks were used based on soil's suitability for wastewater treatment and on its physical, biological and chemical characteristics as discussed in chapter two. Charcoal on the other hand was used based on its adsorption properties as discussed. However, due to environmental concerns, it was preferred that charcoal from agricultural waste materials be used rather than burning charcoal from trees. Maize cob was adopted because of its availability at the time of the research as well as its porous nature.

3.3.1 Brick material preparation

The brick materials were obtained in their block forms of sizes 225mm by 110mm by 75mm. These were then manually crushed to small particles using a hammer. The crushed material was then passed through a BS 410-1 sieve screens of sizes ranging between10.00mm-4.75mm (Itayama et al., 2006; Kondo et al., 2011). Using bigger particle sizes do give lower treatment efficiencies due to the high rate of wastewater flow through the media. In contrary, smaller particle sizes are highly efficient. However, they are prone to frequent clogging especially when the wastewater is heavily loaded with organics. This formed the basis for which particle size range of 10.00mm-4.75mm was chosen to give relatively high treatment efficiency and at the same time avoid unnecessary clogging of the system since the wastewater used in the research was kitchen wastewater with high organic load.

3.3.1.1 Brick particle size analysis

The particle size in this case was analysed by carrying out sieve analysis from which the effective diameter d_{10} was obtained to represent the grain size. The size distribution was determined by the uniformity coefficient U_c.

Effective diameter (d_{10} or d_e): This is the size of the sieve opening through which 10% (by weight) of the particles pass.

Uniformity coefficient (U_c): Is defined as the relationship between the effective diameter and the size of the sieve opening through which 60% (by weight) of the particles pass (d_{60}). Therefore, U_c is expressed as;

$$U_c = \frac{d_{10}}{d_{60}} \qquad \text{Equation 3.1}$$

The values obtained for d_{10} and d_{60} were d_{10} = 4.5, d_{60} = 7.9. From these, Uniformity coefficient was calculated as follows.

$$Uc = \frac{4.45}{7.9} = 0.56$$

The Uniformity Coefficient of 0.56 actually demonstrated a narrow range of particle sizes ranging from 10mm to 4.5mm as required for this research. This was necessary for relatively low flow of wastewater through the media for higher treatment efficiency.

3.3.2 Charcoal preparation

The charcoal used in this research was prepared from maize cob. The research was carried out during a maize harvesting period and a lot of maize cob was available at the moment. The maize cob was therefore acquired from farmers around the university. To drive out moisture from the cobs and for ease of size reduction and burning, they were dried in the sun for about two days. Size reduction was very important before burning because after burning, it would be very difficult to reduce the size through crushing since this would break the charcoal into very small particles that would be easily carried away by the flowing wastewater through the media due to its light weight and at the same time would result into unnecessary clogging of the system.

A pulveriser mill for preparing animal feeds was used in reducing the cob sizes to the desired sizes of 12.5mm and below. After reducing the cob sizes, the charcoal production process began using a muffle furnace in the pyrolysis of the maize cob material at a temperature of 600^0c under airtight conditions for about two hours to maximize on the pore space distribution (Li J.-h., et al., 2014; Gai et al., 2014; Clough et al., 2013). It took about 4 weeks to produce enough charcoal for use in the study since the muffle furnace was small and producing small quantities of the charcoal at every on time. After the charcoal production, sieve analysis was conducted same as for brick material.

3.3.2.1 Charcoal particle size analysis

Using the same sieve standard used for analysing brick material, charcoal particle sizes were characterized and analysed. Based on its highly porous nature, particle size ranging from 12.5mm-0.075mm was used. The smaller sizes in this case were accommodated to provide for larger surface area volume ratio to allow for adoption of more pollutants on the surfaces. The porosity of the charcoal would also allow wastewater to flow despite their small sizes and also the ratio of charcoal in the treatment media was expected to be small.

The two materials were then mixed in the ratio of 1:3 (Oladoja et al., 2006) for charcoal and bricks respectively to form a treatment media which was thoroughly cleaned using clean running tap water to remove dust and other volatile materials which would lead to clogging and highly turbid treated effluent. The processes involved in the treatment media preparation are highlighted by a use of flow diagram in figure 3.2.

Figure 3.1: Flow diagram for treatment media preparation

3.4 Design and Fabrication of the Treatment Unit

3.4.1 Design of the system

The key design parameters considered in this research are the Design Flow Rate, Hydraulic Retention Time, and Design Volume. The total water consumption in a residential building is the water used for various purposes like kitchen, bathing, wash basins, among others. The wastewater generated from all this gives the design volume. This can be calculated separately for black water and grey water. Given that this research was a small-scale pilot project, per capita wastewater generation rate in Kenya per day was used in calculating the design volume. In Kenya, the per capita water consumption per day is averagely 80 litres. After the consumption, 80% of this is generated as waste and in the case of the rural communities, this waste is entirely in form of greywater since in these communities, pit latrines are generally used and therefore no black water is generated from toilets. Therefore, of the 80 litres consumed per person per day, the greywater generated was calculated to be about 64 litres/capita/day. This is the design volume used to generate the design flow rate.

The design flow rate together with the hydraulic retention time are very important in sizing the treatment media chambers. From the design volume of 64litres/day, the design flow rate was obtained to be 44.4ml/min. An adjustment to 50ml/min was considered since this is close to 44.4ml/min. For effective treatment of wastewater through the filter media, a residence time of 16 hours was chosen given the small size of the treatment system (University of Minnesota, 2011). It is however noted that the longer the wastewater takes in the filter media, the better it is treated. However, if longer time is allowed, the capacity of the system reduces, or a bigger system would be needed which might be uneconomical.

From the retention time of 16 hours and the flow rate of 50 ml/min. The treatment media volume was determined to be,

$$Volume = Flow\ rate\ \times Retention\ Time \qquad \text{Equation 3.2}$$

$$Volume = \frac{50ml}{min} \times (16hrs \times 60mins) = 48litres.$$

From this, it was realized that a treatment media of a minimum capacity of about 0.048 M^3 was required for adequate treatment of the intended greywater. Based on previous designs (Itayama et al., 2006; Kondo et al., 2011; Ushijima et al., 2013) the treatment media is placed in a number of trays above each other, whose total volume should allow for at least 16 hours of contact time between the media and the wastewater rather than one big tray of the same volume. This is to maximize on the surface area for provision of oxygen used for aerobic degradation of organics taking place on the upper portions of the treatment media chambers. One big tray limits this process since the surface area of exposure is limited and also a large space will be needed for such system. The allowable dimensions for such trays used are as shown in Figure 3.2.

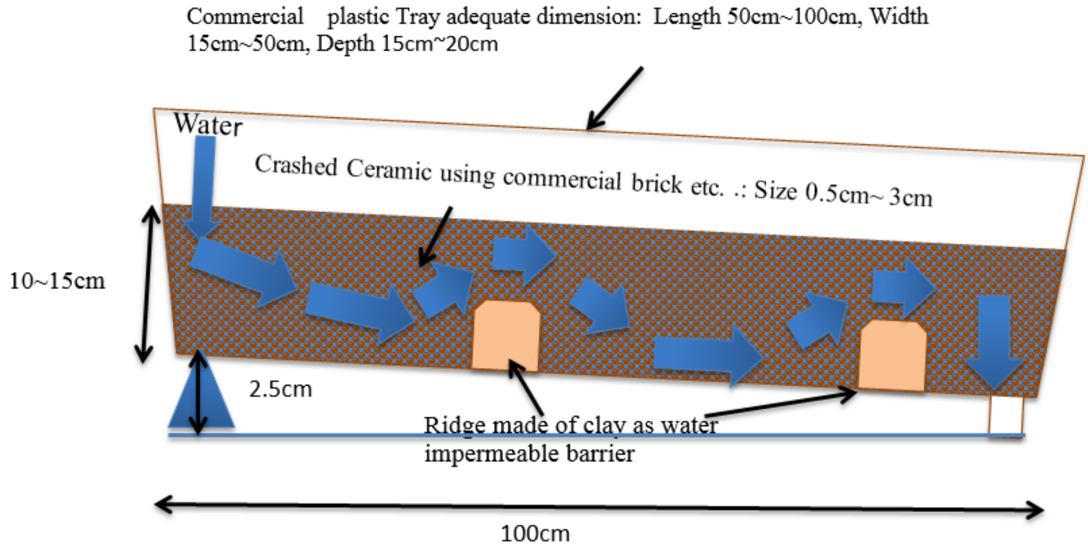

Figure 3.2: Tray dimensions for on-site wastewater treatment systems (Itayama et al., 2006)

Based on the above dimensions, it would be convenient to obtain commercial trays with measurements falling within the given ranges rather than fabricating the same since fabrication would be costly and full of challenges such as leakages, difficulty in getting plastic sheets and joining materials. After doing thorough survey, a decision was finally reached to use trays obtained from the market with the shape and measurements indicated in Figure 3.3. The trays were then fitted with two ridges to act as buffers at 0.2 metres apart to divide the trays into equal parts to avoid linear flow of the wastewater through the media. The wastewater would therefore have to flow below and then above the subsequent ridge as indicated in the side view of the tray diagram (figure 3.4). This was done to avoid ponding of the water at the bottom of the trays which would occur if the wastewater was made to flow above the two ridges and at the same time to prevent flow short circuiting that would shorten the retention time. Through such arrangement, the wastewater would come into contact with almost the entire volume of the filter media material.

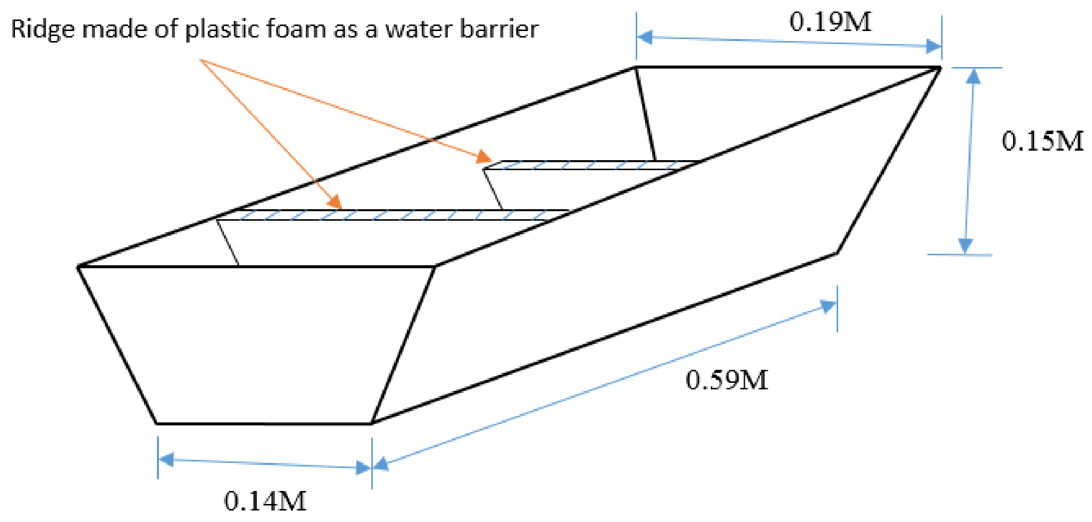

Figure 3.3: A sketch of a tray used in the research, showing its dimensions and ridges for buffer flow

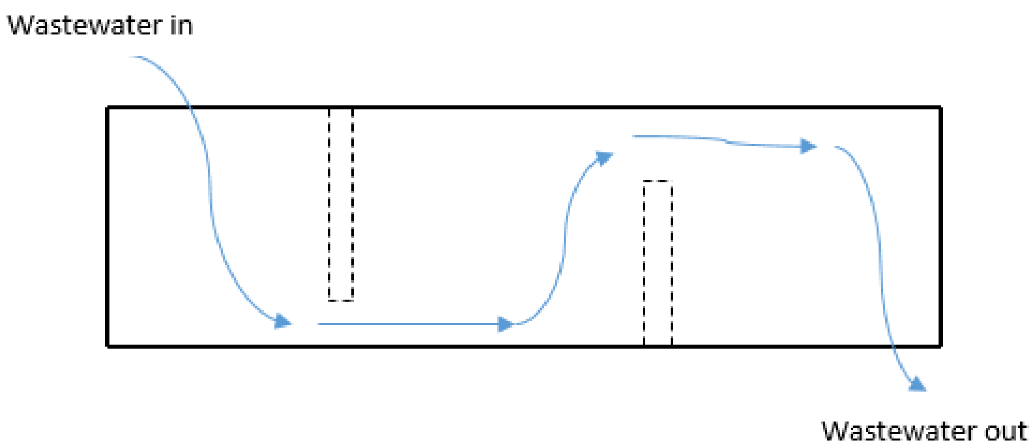

Figure 3.4: A sketch of the side view of the trays used in the research showing how the ridges would influence the wastewater flow through the treatment media

The type of trays provided for dimensions falling within the recommended ranges as indicated Itayama et al. (2006). From the dimensions, the capacity of a single tray was then determined to be about $0.014M^3$. A decision was then reached to use five of such trays stalked above each other in each of the two systems which would hold $0.07M^3$

treatment media capacity. This if compared with the 0.048M^3 media capacity calculated above would provide for longer hydraulic retention time of about **23 hours** which was to the advantage of the study over the 16 hours selected. With this, a very good treatment result was anticipated. The trays were set up for the two systems and the expected capacity of the two systems was 140 litres of greywater to be treated per day.

3.4.2 Fabrication of the whole system

Having prepared the treatment media and obtained the trays for holding the treatment media materials, five trays were then stacked above each other in each of the two systems just as explained above. Fabrication of two metallic racks for the two systems was done in the Department of Mechanical and Production Engineering in Moi University. The racks were designed in such a way that they would provide for a slope of 2.5% to each tray arranged vertically above each other in a counter direction to provide for gravitational flow of wastewater through the treatment media from one tray to the next below it in an alternating direction (Itayama et al., 2006). The trays were then fitted with drain holes fitted with plastic tubes at one bottom end for draining the wastewater into the next tray below it.

3.4.3 System set-up

One of the two systems was used as a control and its treatment chambers were filled with brick material alone. The other system which was an improvement of the control had its treatment chambers filled with bricks mixed with charcoal at a ratio of 3:1 respectively (Oladoja et al., 2006).

The establishment of the two systems involved setting up five components each including wastewater collection point, water holding tank, anaerobic chambers, filter beds (bioreactor chambers) and treated water collection basins. The first two components - wastewater collection point and holding tank were shared by the two systems. The wastewater was collected from one source and then filled into one holding tank. The remaining components were separate for each system. A layout sketch and the actual set up of the two systems are illustrated in figure 3.5 and plate 3.1 respectively.

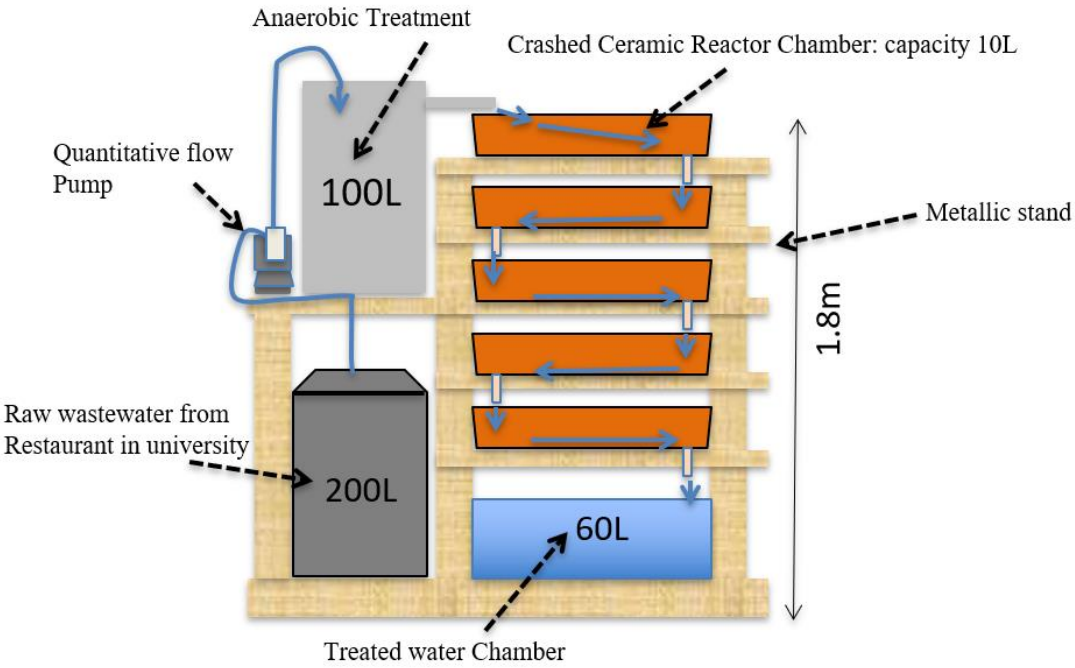

Figure 3.5: Schematic flow diagram for wastewater treatment unit set up for this research

Plate 3.1: A photograph of the study set up in the University laboratory.

3.5 Wastewater Collection Point

Given that the wastewater meant for this research was being collected from a student cafeteria within the University, an immediate manhole that collects the kitchen wastewater from the cafeteria was identified. This manhole was preferred because it was tapping the wastewater before mixing with the black water from the cafeteria and student hostels. The manhole was initially cleaned of all the sludge that had settled in it to allow for fresh wastewater to start accumulating in it for daily collection.

As previously discussed, a total of about 140 litres of raw wastewater was collected from the manhole daily for treatment. This was done by blocking the outlet of the manhole every morning for the wastewater to accumulate then fetched manually into 20 litres containers and transported to the laboratory using a wheelbarrow.

3.6 The Holding Tank

In the laboratory, the raw wastewater was put in a 165-litre plastic holding tank. It was considered that one holding tank of such volume was enough to supply the two systems since each system had a capacity of treating about 70 litres per day totalling to about 140 litres for the two systems and therefore a holding tank of 165 litres was adequate for the intended purpose. To be sure of homogeneity and non-settlement of solids in the holding tank, the raw wastewater was consistently stirred using a wooden stick. Also, to stop biological degradation of the wastewater from taking place in the tank, some ice cubes were immersed every morning into the holding tank to keep the holding tank temperature below 5^{0}C to deactivate any bacterial action on the wastewater. This was done because the holding tank was not meant for any form of treatment.

3.7 The Anaerobic Chambers

The third component in the two systems were the anaerobic chambers. This is where the treatment of the wastewater would start. The anaerobic chambers were two in place with a capacity of 165 litres each. These were made from plastic tanks already available in the market. Some modifications were done on the tanks to ensure that they provided for suitable conditions for anaerobic processes. Three holes were drilled on the tanks for inflow, outflow and overflow. The anaerobic chambers were meant to hold a volume of about 100 litres each at any given time during the treatment process. This would provide for an adequate retention time necessary to help in reducing the organic load of the wastewater by about 50% through settlement of the solids at the bottom of the tanks and also through anaerobic degradation of the organics by anaerobic bacteria which would thrive well due to prevailing anaerobic conditions in the chambers (Itayama et al., 2006; Kondo et al., 2011). To provide for the volume, 100 litres of water was poured into the tanks and overflow points created at the water level. The holes were then fitted with return overflow pipes to the holding tank such that once the water reaches that level, it flows back to the holding tank. Just slightly above the overflow points, inlet holes were drilled. These were strategically placed slightly above the overflow such that once the anaerobic tanks are filled to the required 100 litres capacity, the incoming fresh raw water would flow back immediately to the holding tank through the overflow spout created at the point it drops into the anaerobic tank without mixing with the already degrading water in the chambers. The overflow pipe was also crucial in allowing water to flow throughout without stopping the pump as it helped maintain the capacity of the chambers at 100 litres and the excess would flow back to be pumped again (water circulation). This allowed the system to run throughout the day and night without monitoring the volume and topping up the anaerobic chambers. Once the holding tank was filled to its capacity, it would run for 24 hours with the circulation of water in place. The outflow pipe that directed wastewater out of the anaerobic chambers to the filter beds was placed about 0.25 metres above the bottom of the anaerobic chamber tanks. This was to reserve some volume (dead volume) for settling of solid materials at the bottom of the chambers. This was very essential since these materials would block the control valve fitted on the outlet tube for metering the flow rate of wastewater from the anaerobic chambers into the filter beds in case the outlets were to be placed at the bottom of the tanks. To reduce the amount of settled materials, a valve was fitted at the bottom of the tanks for occasional desludging of the anaerobic chambers once it was noted that there were a lot of solids flowing into the treatment media chambers from the anaerobic tanks causing blockages of the valves and clogging of the media chambers.

To ensure that the conditions in the anaerobic chambers were actually septic and fully anaerobic, the anaerobic tanks were filled halfway with charcoal which was suspended using wire mesh above the dead volume. The charcoal used in this case were commercial and were not subjected to any condition during preparation. The charcoal was used to help in eliminating any oxygen which might still be remaining in the water in the chamber. Charcoal is good in adsorbing oxygen on its surfaces and its porosity can help it adsorb more.

To provide for gravity flow of the wastewater from the anaerobic chambers through the filter beds, the anaerobic chambers were placed on 1.8 metres high metallic stand and the anaerobic tank was 0.5 meters higher than the tray rack that held the filter beds. Because of this height, a quantitative flow pump was fitted to hoist the raw wastewater from the holding tank into the anaerobic tanks.

3.8 The filter beds

The filter beds provided for further treatment of the wastewater. In these beds, there was a combination of both aerobic processes in the upper sections of the filter materials and some anaerobic processes at the bottom where oxygen could not access. Just as mentioned earlier, the filter bed was made from plastic trays each filled with 23.5kilgrams of treatment media. The treatment media in this case were crushed building bricks for the control system and a mixture of the crushed bricks and charcoal made from maize cob for the system studied. Five of such trays were stalked above each other in a metallic rack which would provide for a flow gradient of 2.5% to enable gravitational flow of wastewater from one tray to the next below it in a counter direction (Itayama et al., 2006). The flow from the anaerobic chambers into the filter beds was checked daily using a measuring cylinder and a stop timer to ensure that it was always 50Ml/min.

3.9 Effluent Collection

To collect treated water, 60 litre basins were placed below the two racks to collect water draining from the last trays on the racks. This was the water regarded as treated and analysed for water quality parameters. The basins were regularly emptied to provide volume for the flowing water from the two systems.

3.10 Sampling and Test Procedures

To allow for adequate time of degradation at the anaerobic chambers and due to the number of parameters to be analysed, sampling was done once a week, except in cases when there were interferences such as power blackouts and the cafeteria not operational. Any interference prompted frequent sampling to check on the effect on the performance of the system. The sampling points were the raw wastewater fresh as it was brought from the cafeteria, the outlet of the anaerobic chamber which fed the filter bed and the outlet of the filter bed. The water quality analysis was done for the parameters such as:

- *Biological Oxygen Demand (BOD_5)*
- *Ammonia-Nitrogen*
- *Nitrite-Nitrogen*
- *Nitrate-Nitrogen*
- *Total Nitrogen*
- *Phosphate-Phosphorus and*
- *Total Phosphorus*

For all the water quality parameters mentioned, use was made of the recommended American Standard Methods for the Examination of Water and Wastewater (APHA-

AWWA-WEF, 1998). The measured values were compared with the standard requirements for the same as documented by WHO guidelines for wastewater reuse.

Determination of the treatment efficiencies

The efficiency for any kind of treatment system is commonly expressed in terms of 'percentage removal' defined as:

$$\%R = ((C_i - C_e)/C_i) \times 100$$

Equation 3.3

Whereby:

$\%R$ = percent removal

C_i = influent concentration

C_e = effluent concentration

This forms the basis on which the removal efficiencies for the systems were evaluated. For the parameters in question i.e. Biological Oxygen Demand, Nitrogen and Phosphorus, the measured values between two reference points' i.e. influent and effluent conditions were compared. This was done by evaluating the effluent condition as a percentage of the influent for the different sections, that is, the raw wastewater, anaerobic chambers and the filter beds.

3.11 Statistical Analysis

Three major pollutants were picked for the statistical analysis, which are Biological Oxygen Demand (BOD_5), Total-Nitrogen (T-N) and Total Phosphorus (T-P). Given that the two systems provided two independent data sets, a regression analysis was conducted to determine if there was any correlation between the two data sets. A student’s t-test was then conducted to determine if there was any significant difference in the means of the two data sets.

CHAPTER 4: RESULTS AND DISCUSSION

This chapter presents the findings of the wastewater treatment and reuse system studied. The performance was determined for the three sections of the system, that is; the anaerobic chamber and the two slanted filter bed chambers by way of their removal efficiencies for the various selected water quality parameters. It specifically presents a comparison of the overall performance of the filter media prepared from crushed bricks when amended with charcoal in comparison to that prepared from crushed bricks alone which was used as the control.

The results presented here were obtained through physical observation and chemical analysis.

4.1 Physical Observations

Through observation, there was a significant distinction between the raw wastewater and the two effluents from the treatment systems. The raw wastewater had a grey colour with a lot of food waste in solid form, some of which were, floating, some suspended and some could settle at the bottom of the holding tank due to their sizes and weight. In the holding tank the settlement was inhibited through constant stirring since all these wastes were to be transferred to the anaerobic chambers. In some circumstances, there was foul smell noticed from the raw wastewater. This was an indication of degradation taking place in the manhole from which the wastewater was collected due to long time (overnight) of stay in the hole.

From the anaerobic chambers, the effluents were dark in colour with some suspended particles which were also dark. The larger particles were not present in these effluents. The dark colour coupled with the foul smell of a rotten egg was an indication of anaerobic processes present in the anaerobic chambers. The absence of large particles of organic waste also proved that there was settlement of these solid organic matter in the anaerobic chamber.

The effluents from the filter beds were clear from the particles (Plate 2a). However, the effluent from the brick system was constantly having brown colour as observed daily (Plate 2b). Also some smell could still be felt from the effluent. This is a show of incomplete treatment process in the system. Contrary to the effluent from the brick filter beds, the effluent from the brick and charcoal filter beds was observed to be very clear with no smell coming from it (Plate 2c). The differences between the raw wastewater and the effluents from the bricks and bricks plus charcoal systems are illustrated in Plate 4.1.

Plate 4.1: Illustration on the observed differences between raw wastewater and the two effluents

4.2 Chemical Analysis

To ascertain the actual level of treatment of the wastewater by the two systems, a close check of the performance was necessary through chemical analysis for the raw wastewater, the effluent from the anaerobic chambers and the effluent from the bioreactors as presented hereunder.

4.2.1 Raw greywater quality

The greywater used in this research was sourced from the student cafeteria in the University and was loaded with organic wastes from kitchen. Chemical analysis was done on selected parameters to determine the initial levels of these parameters before the greywater was subjected to any form of treatment. The results obtained are presented below in tables and charts and discussed in detail.

4.2.1.1 Biological Oxygen Demand (BOD_5) in Raw Greywater

Variations in the BOD_5 concentrations in the raw wastewater was observed ranging from 374mg/l to 732mg/l, with an average of 544mg/l (Figure 4.1). These values were however very high compared to the (NEMA, 2006) guidelines of 30mg/l BOD_5 for discharge into the environment and even 500mg/l of BOD_5 for discharge into public sewers. A proper system was hence required to reduce the BOD5 to levels below the recommended levels in the guidelines.

The observed variations could be due to the washing process which was taking place during the time of collection. Rinsing generally uses a lot of fresh water and this would dilute the wastewater leading to low levels of water quality parameters. The BOD_5 in the raw greywater used in the research is illustrated in the figure 4.1, showing varying concentrations during the 22 weeks of the study.

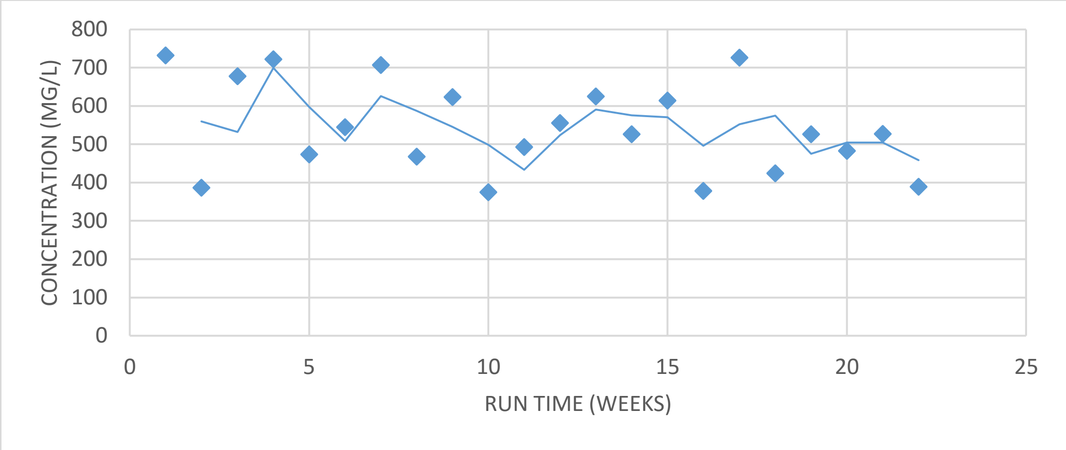

Figure 4.1: Recorded BOD5 concentration in raw greywater

4.2.1.2 Total Nitrogen (T-N) in Raw Greywater

The raw greywater also recorded varying values of Total Nitrogen concentrations as in Figure 4.2. These were ranging between 23.62Mg/l and 43.93Mg/l, with an average of 32.65Mg/l and were still within the range of 0.6 to 74 Mg/l provided by Dalahmeh (2013). The presence of Total-Nitrogen in the raw greywater was due to the organic load from food waste generated in the kitchen. These were in the form of both organic nitrogen (Proteins, peptides and Amino acids) and inorganic nitrogen (NH_4, NO_2 and NO_3).

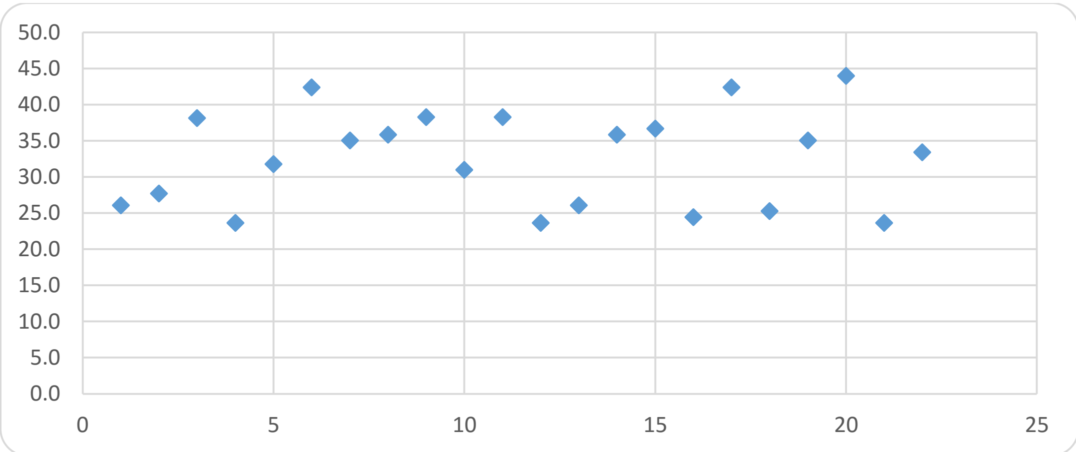

Figure 4.2: Recorded Total Nitrogen concentration in raw wastewater.

4.2.1.3 Total Phosphorus

From the findings, the average concentration of Total Phosphorus in the raw wastewater was 6.5Mg/l (Figure 4.3). This was within the range given in the literature that Total Phosphorus in greywater ranges between 4-14 Mg /L in non-phosphorus detergent greywater and 6-23 Mg/L in areas where phosphorus-containing detergents are still in use (Dalahmeh, 2013).

The other parameters analysed in the raw wastewater were Ammonium ions, Nitrites, Nitrates and Phosphates. These parameters were found to be within the ranges provided in the literature with some slight variation in Total Nitrogen concentrations. For instance, the concentration ranges in mg/l recorded by (Huelgas et al., 2009) in raw Kitchen wastewater for Total Nitrogen (TN), Ammonium Nitrogen (NH_4-N), Nitrate Nitrogen (NO_3-N) and Total Phosphate (TP) were 21.9–43.5, 0.3–2.7, 0.9–5.3 and 2.9–14.5 respectively. This showed that the data obtained was not off the target value. It is however worth noting that the Total Nitrogen concentrations in this study was higher than Ammonium, Nitrites and Nitrates values (Table 4.1). This could suggest that more Nitrogen was available in the raw wastewater in the organic form as compared to the inorganic forms. The presence of the inorganic forms of nitrogen in the raw wastewater had an indication that the wastewater was already undergoing degradation in the manhole leading to transformation of the organic nitrogen into the inorganic forms.

Table 4.1: Range of raw wastewater quality (Source: Field data, 2015)

Parameter	**Range (Mg/l)**	**Average Mg/l)**
BOD5	374.6 to 732.2	544.3
T-N	23.6 to 44.0	32.7
NH4	0.7 to 1.8	1.2
NO3	0.5 to 1.3	0.8
NO2	0.029 to 0.057	0.038
TP	4.09 to 8.4	6.5
PO4	1.1 to 3.1	2

4.2.2 Treatment in the anaerobic chambers

The parameters analysed were the same as those analysed in the raw wastewater in order to find the level of their reduction after the first step of treatment in the chambers, especially BOD_5 which studies indicate can be reduced by about 50% in the anaerobic systems (Itayama et al., 2006). The results obtained for every parameter analysed are discussed as follows.

4.2.2.1 BOD_5 removal in the anaerobic chambers

There was a significant drop in the quantities of BOD_5 leaving the anaerobic chambers as compared to what was coming in (Table 4.2).

Table 4.2: BOD5 Concentrations in the Raw Wastewater and the effluent from the anaerobic chambers

Section of the system	BOD_5 Range (Mg/l)	Average BOD_5 (Mg/l)	Reduction rate (%)
Raw Wastewater	374.6 - 732.2	544.3	0
Anaerobic Chamber 1 (Study system)	162.6 - 294.3	235.1	54.3
Anaerobic Chamber 2 (Control system)	144.3 - 310.3	227.3	55.9

This was a show of great performance of the chambers in terms of degradation of the organics in the wastewater. The level of reduction matched those reported in the literature material as about 50% reduction of BOD_5, (Dawes, 2006). This justified their inclusion in the systems. The performance of the two chambers were there about the same, giving averages of 54.3% and 55.9%, falling within the range of 40% to 60% level of reduction of BOD_5 provided in the literature. It was observed that the BOD_5 was very high in the Raw Wastewater, with the highest value of 732.2Mg/l given the fact that the waste was entirely kitchen Wastewater. Nonetheless, the anaerobic chambers performed satisfactorily well despite this fact, reducing these concentrations to the lowest level of 144.3Mg/l, giving the highest level of reduction of 80%. It was also worth noting that the operation of the chambers started relatively lower in the first four weeks. This consistently rose to the highest of 77% in the anaerobic chamber 1 (System for study) in the 7th week of operation and 80% in the anaerobic chamber 2 (control system) in the same week of operation. This may be attributed to the fact that the system took about 28 initial days to mature. During this period, the performance was still low since the micro-organisms (anaerobes) needed for the anaerobic degradation of the organics were not fully grown (Nnaji et al., 2013). By the 7th week, the system was fully mature giving the best performance thus their highest efficiency levels. Figure 4.3 shows the trend lines in the performance of the anaerobic chambers in terms of per cent BOD_5 reduction for the entire study period.

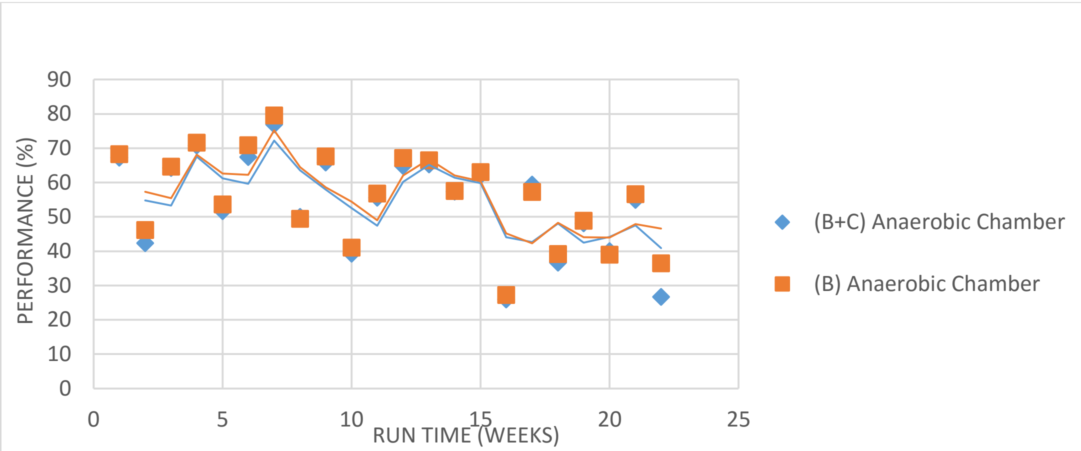

Figure 4.3: Trend lines in the performance of the anaerobic chambers for BOD5 reduction

By the end of the study period in the 23rd week, the performance of the anaerobic chambers had gone down to as low as 27% for the anaerobic chamber 1 (study system) and 36.5% for the chamber 2 (Control System). At this point the system could be assumed to be over mature meaning that there was over population of the microbes in the system leading to their death due to intoxication. At this point, the system is said to have reached its run time end and needed maintenance through replacement of the charcoal material or cleaning the materials with clean water and drying them in the sun for a long period of time to acquire their natural state.

It is quite interesting to note from the plot that the two anaerobic chambers showed a similar trend in their performances throughout the study period. As discussed in chapter three, the anaerobic chamber assists in treating wastewater through retention of any settle-able organic matter at the bottom of the chambers as well as anaerobic breakdown of the organics into the inorganic forms, especially into Ammonium-Nitrogen (NH_4-N) form. The anaerobic breakdown of the organics takes place in the presence of anaerobic bacteria (anaerobes) which thrives well only in the absence of oxygen or in very low oxygen conditions. This process of converting organic Nitrogen into inorganic forms in anaerobic conditions is termed as **denitrification process**. This is the initial process in any wastewater treatment and is very essential. To enhance the anaerobic processes in this research, charcoal was added in the chambers to eliminate by adsorbing any soluble oxygen that would be present in the wastewater and would cause any aerobic situations in the chambers.

Worth mentioning is the colour of the effluent from the anaerobic chambers which was dark with foul smell of a rotten egg. The smell indicated that there was production of gases such as methane (CH_4) and carbon dioxide (CO_2) normally produced during any anaerobic process. This was an indication that the systems were well in operation leading to their good performance.

Generally, the anaerobic chambers were very critical in terms of BOD_5 reduction as indicated by more than 50% reduction in the parameter in both cases.

4.2.2.2 Nitrogen removal in the anaerobic chambers (Total Nitrogen, Ammonium-Nitrogen, Nitrite-Nitrogen and Nitrate-Nitrogen).

In raw wastewater, nitrogen exists in its organic forms i.e. (proteins, amino acids and peptides). However, presence of some inorganic forms like (NH_4, NO_3 and NO_2) in the raw wastewater shows that the wastewater is already undergoing degradation. The onset of the brake down of the organic nitrogen is in the anaerobic chambers. It is here that the organic nitrogen is converted to Ammonium-Nitrogen through denitrification process by anaerobic bacteria. It requires absolute exclusion of oxygen for this process to occur. Other forms of inorganic nitrogen such as Nitrate-Nitrogen and Nitrite-Nitrogen may also undergo this process if they were present in the raw wastewater. If traces of these inorganic nitrogen (nitrates and Nitrites) are found in the effluent of the anaerobic chambers, it is a clear indication that the system is not fully anaerobic.

During the study, the effluent from the anaerobic chambers were tested for Ammonium, Nitrates, Nitrites and Total Nitrogen and the result presented in table 4.3 against the values obtained from raw wastewater showing their levels of reduction.

Table 4.3: Nitrogen pollutants in raw wastewater and effluent from anaerobic chambers

System Section	**Parameter**	**Range (Mg/l)**	**Average Mg/l)**	**Reduction Rate (%)**
Raw Wastewater	NH_4	0.75 - 1.84	1.26	-
	NO_2	0.02 - 0.06	0.04	-
	NO_3	0.54 - 1.31	0.78	-
	T-N	23.62 - 43.98	32.65	-
Anaerobic Chamber 1 (Study system)	NH_4	0.88 - 1.97	1.6	-27
	NO_2	0.02 - 0.04	0.03	25
	NO_3	0.31 - 1.00	0.62	20.5
	T-N	3.26 - 12.22	7.11	78.22
Anaerobic Chamber 2 (Control system)	NH_4	1.14 - 2.04	1.65	-31
	NO_2	0.02 - 0.04	0.03	25
	NO_3	0.08 - 1.07	0.6	23.1
	T-N	4.07 – 10.6	7.03	78.46

It was observed from the results that there were some ammonium present in the raw wastewater, an indication that the raw wastewater was already decomposing from the collection point. However, the effluent from the anaerobic chambers showed increase in the concentration of the ammonium nitrogen. This showed an increase of 31.3%

averagely in the anaerobic chamber for the study system and 35.7% in that for control. This is enough to justify that the chambers were providing the anaerobic conditions necessary for denitrification processes leading to the conversion of the organic nitrogen in the raw wastewater fed into the chambers to NH_4^+-Nitrogen. However, the percent increase in the ammonium concentration was law since raw wastewater already had a lot of ammonium due to prior decomposition of the raw wastewater in the collection point.

The trend lines for the Ammonium-Nitrogen in the wastewater and the effluents from the two anaerobic chambers are as shown in figure 4.4.

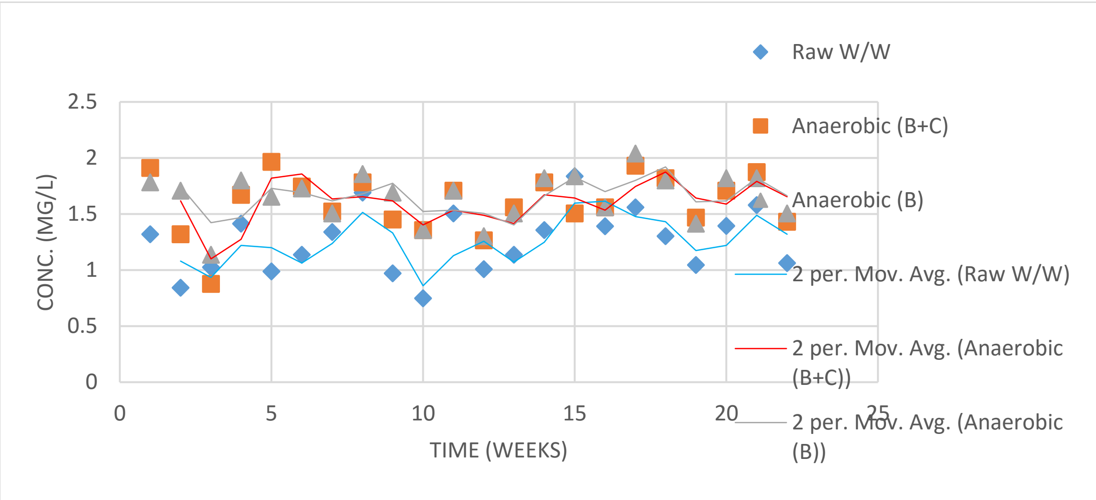

Figure 4.4: Trend lines for ammonium concentrations in the Raw Wastewater and the effluents from anaerobic chambers

The chart indicates that ammonium was lower in raw wastewater than in the anaerobic chambers during the entire period of study. The operation of the two chambers followed a similar trend with some very minor variations in some few days especial in the onset of the study period. The variations occurred due to some technical challenges experienced during the research period. Some of these challenges include the blocking of the outlet valves of the chambers by large particles which would want to exit the chambers with the effluent. If this was not noticed and corrected in time, it would mean that the retention time in that particular chamber would be prolonged compared to the other leading to further refining of the wastewater. This would mean that more Ammonium Nitrogen would be produced in that chamber than the other.

Also present in the raw wastewater were nitrites and nitrates. These were present in low concentrations and after being passed through anaerobic conditions, their quantities reduced further by about 20%. This was also because of denitrification processes in the anaerobic chambers which converted the nitrates and nitrites into nitrogen gas. The presence of these pollutants in the raw wastewater was a justification that the wastewater was not actually raw since it was already undergoing decomposition. Also worth mention is the fact that as much as there was denitrification processes in the chambers, this process was not complete as there were some traces of nitrites and nitrates in the effluent from these chambers which could have been converted fully into nitrogen gas if the process was complete. This could be because of lack of carbon material which would provide energy in for the process to be complete.

Anaerobic process is very essential in the removal of nitrogen pollutants in any wastewater. It starts the process by transforming the organic nitrogen into Ammonium-nitrogen through denitrification processes in a complete exclusion of oxygen (Loomis., 1999) and also converting the other inorganic forms (Nitrites and Nitrates) which might also be present in the wastewater into nitrogen gas. The conversion of the Nitrites and Nitrates will only take places if there is a carbon source of energy in a complete exclusion of oxygen.

Figures 4.5 and 4.6 illustrate the trends in the performance of the anaerobic chambers in terms of percent reduction of Nitrite and Nitrate-Nitrogen in the raw wastewater during the study period.

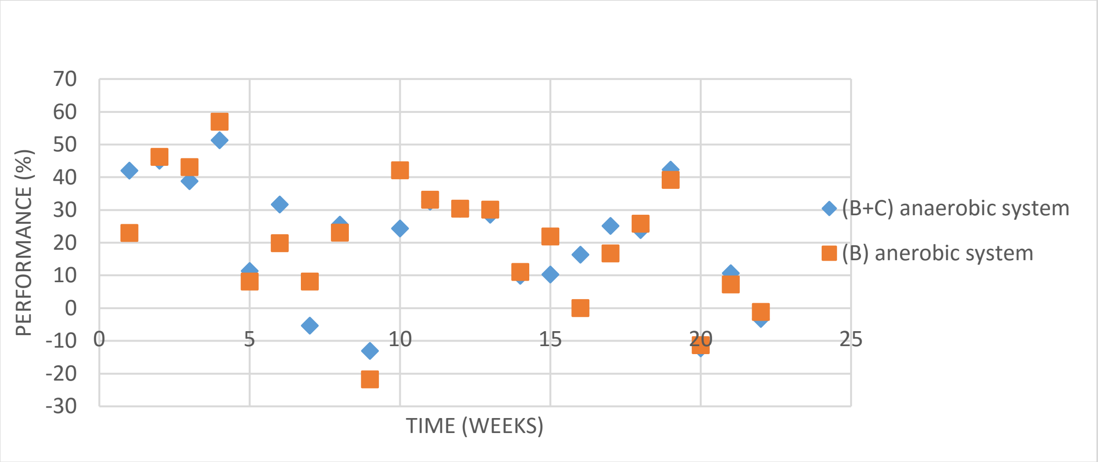

Figure 4.5: An illustration of the performance of the anaerobic chambers in terms of NO2 removal

The performance of the anaerobic chambers were well above 10% in NO_2 removal. As discussed earlier, the low performance is attributed to lack of carbon in the chambers to act as a source of energy. At times the performance would indicate negative percentage implying that NO_2 were recorded in the anaerobic chambers' effluents than in the raw wastewater. Such a scenario would arise because of the washing process which was taking place during the raw wastewater collection. As discussed earlier, if the water was collected when rinsing was taking place, the raw wastewater would have very low levels of pollutants and if the same is used for analysis, the concentration levels will be lower than those from the anaerobic chambers. The average performance of both chambers was about 20% removal of the Nitrite-Nitrogen. This was an indication of an ongoing processes of denitrification in the anaerobic chambers.

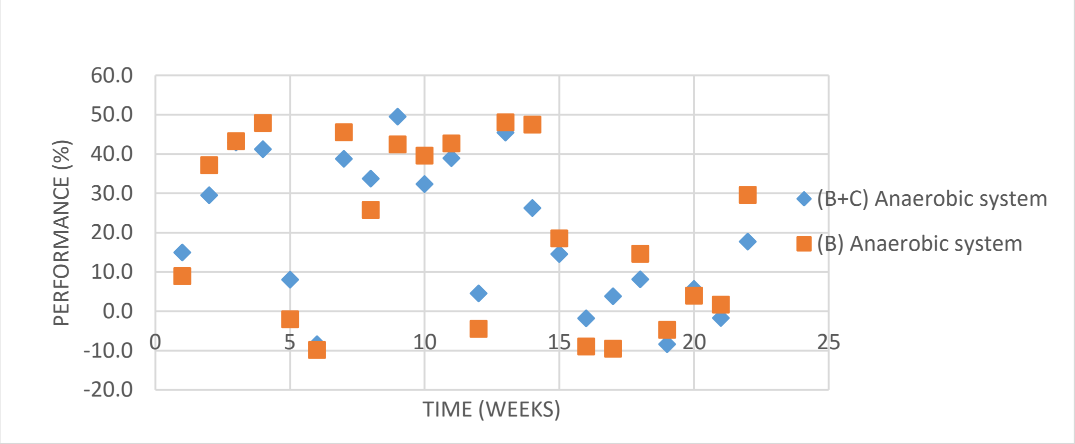

Figure 4.6: An illustration of the performance of the anaerobic chambers in terms of NO3 removal

Removal of the NO_3 in the anaerobic chambers was of the same trend as that of the NO_2 with the average percent reduction of 19.8% in the anaerobic chamber 1 (for study system) and 22% in the anaerobic chamber 2 (for bricks system). A point to note was that the concentrations of NO_3 were a little bit higher in both the Raw Wastewater and the effluents from the anaerobic chambers as compared to the NO_2 concentrations. This is due to the fact that NO_3 is more stable in nature as compared to NO_2.

The form of Nitrogen which was most abundant in the Raw Wastewater was the Total Nitrogen (T-N). This is because Total Nitrogen in Wastewater exists in the form of both soluble and insoluble nitrogen. Total Nitrogen is a term used to refer to all forms of nitrogen in the wastewater (both organic and inorganic nitrogen). These originate from all the organic matter contained in the Raw Wastewater. In case the wastewater is already undergoing decomposition, some of the organic matter is converted to inorganic form of the nitrogen through mineralisation. Given that the raw wastewater used in this research was purely kitchen waste, it had a lot of organic matter from food waste. This provides a good reason why there was a lot of T-N in the Wastewater.

As observed in the previous table 4.3, the concentration of Total Nitrogen in the Raw Wastewater was ranging between 23.6Mg/L to 44.0Mg/L, with an average of 32.65Mg/L. Passing the wastewater through the anaerobic chambers reduced the concentrations to 7.11Mg/L in average for the anaerobic chamber 1 and 7.03Mg/L in chamber 2. This was a great show of performance of both the anaerobic chambers with average per cent reduction of the Total Nitrogen of 77.7% and 77.3% respectively. This is a good justification for why the anaerobic chambers had to be incorporated in the treatment systems.

The trends of operation of the two anaerobic chambers in terms of Total Nitrogen removal in the wastewater is illustrated in figure 4.7.

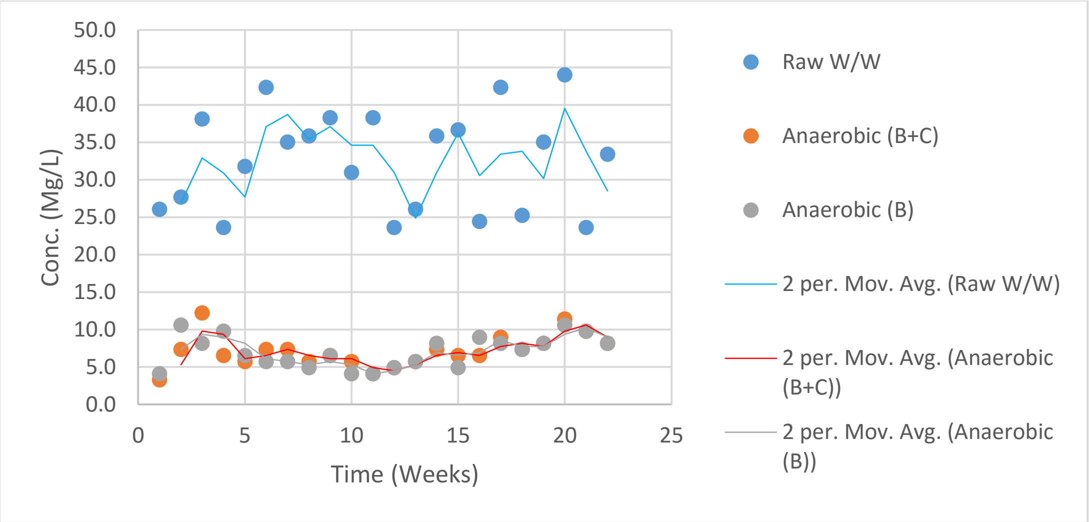

Figure 4.7: Trend lines for T-N concentrations in the Raw Wastewater and the effluents from anaerobic chambers

It is clearly shown in Figure 4.7 that there was a significant reduction of Total Nitrogen concentrations in the Wastewater after passing it through the anaerobic chambers. The T-N concentration in the Raw Wastewater was well over 23Mg/L in the entire treatment period. This was reduced to below 12Mg/l for the entire period for both systems. It is also noted that the two anaerobic chambers demonstrated a similar trend and provided equal levels of treatment in the reduction of T-N pollutants with very minor deviations in the first four weeks. These variations resulted from the fact that the system was still unstable.

To further illustrate the performance of the anaerobic chambers, it was necessary to plot the percentages of reduction of T-N concentrations in both anaerobic chambers for the entire study period as in figure 4.8.

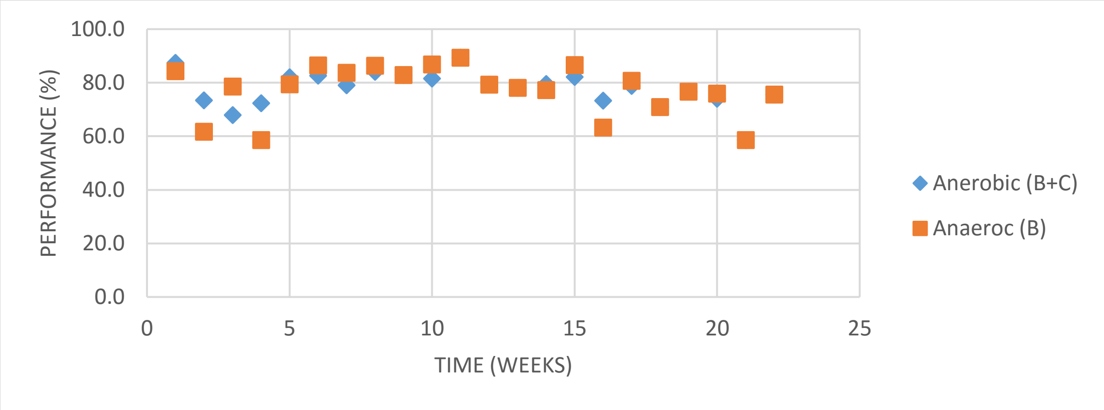

Figure 4.8: An illustration of the performance of the anaerobic chambers in terms of T-N removal

Figure 4.9 shows good performances of the two anaerobic chambers which followed a similar trend for the entire period of treatment. The performance for both chambers was above 60% in most of the analysis done, with quite a number of results giving same level of treatment.

The Total Nitrogen concentration in the Wastewater was reduced in the anaerobic chambers through two main processes. The first was through the settling of the organic matter from food wastes at the bottom of the chambers. This contributed to a major percentage loss as most of the Organic Nitrogen (in solid form) was lost through this process. The second process was through metabolism by the micro-organisms present in the chambers as they fed on these wastes and released them in the form of gasses (CO_2, CH_4).

4.2.2.3 Phosphorus removal in the anaerobic chambers (Total Phosphorus and Phosphate)

Because phosphorus changes form, it is measured in terms of total phosphorus rather than any single form to determine the amount of nutrient that can feed the growth of aquatic plants such as algae. In this study, measurement was done for both Phosphate and Total phosphorus. Phosphate was measured because the wastewater was from kitchen and had a lot of detergents used in the washing of utensils and phosphate is a constituent of such detergents.

The beginning of biological treatment of Phosphorus is in the anaerobic chambers and therefore it was necessary to determine how much of phosphorus was reduced after passing the wastewater through the anaerobic chambers of the treatment systems which were set up. The results obtained after determining the concentrations of Phosphate and Total phosphorus in both Raw Wastewater and the effluent from the anaerobic chambers are in table 4.4.

Table 4.4: Phosphorus pollutants in raw wastewater and effluent from anaerobic chambers

System Section	Parameter	Range (Mg/l)	Average Mg/l)	Reduction Rate (%)
Raw Wastewater	PO_4	1.05 - 3.06	2.02	-
	T-P	4.09 - 8.40	6.51	-
Anaerobic Chamber 1 (Study system)	PO_4	2.04 - 4.44	3.12	-54.46
	T-P	2.58 - 7.03	4.37	32.42
Anaerobic Chamber 2 (Control system)	PO_4	1.92 - 4.68	3.23	-59.9
	T-P	2.94 - 7.10	4.36	32.58

Phosphate is a form of phosphorus whose origin in wastewater is majorly soaps and detergents used in cleaning purposes in households. Given that the Raw Wastewater was from the kitchen, it had a substantial quantities of phosphate concentrations ranging between 1.05Mg/L and 3.06Mg/L, with an average of 2.02Mg/L. This was a little bit lower than those recorded in some studies, for instance (Friedler, 2004) gave a range of 13Mg/L to 49Mg/L. The low levels of phosphate in this research could be due to use of a lot of clean water during rinsing of utensils in the kitchen thus diluting the detergents used. Passing the wastewater through anaerobic conditions increased the phosphate concentrations as witnessed in the effluents from the anaerobic chambers. There was an average increase of 61.6% in phosphate concentration in the effluent from the anaerobic chamber 1 while the effluent from the anaerobic chamber 2 presented an average of 68.3% increase in the phosphate concentration. The trend was observed throughout the study period as illustrated in Figure 4.9.

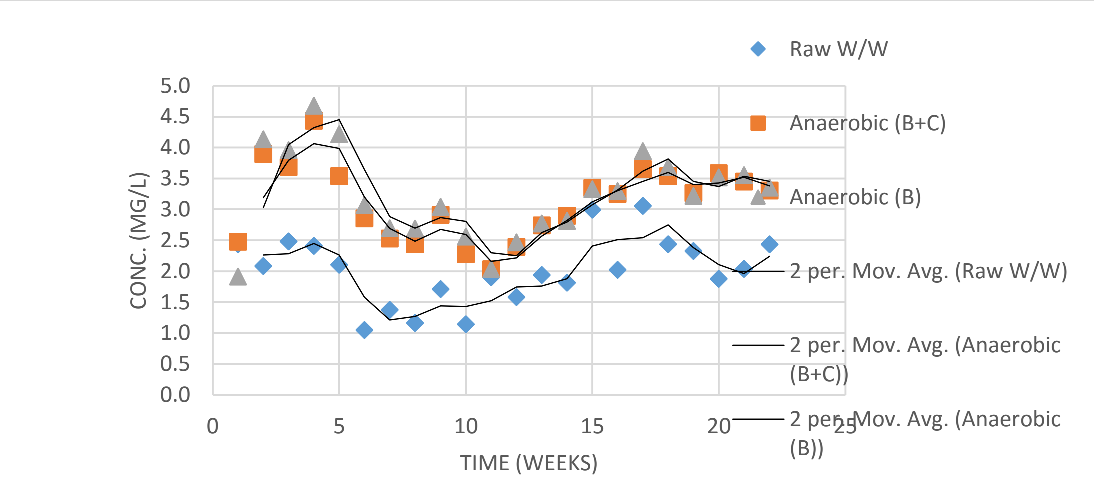

Figure 4.9: Trend lines for Phosphate concentrations in the Raw Wastewater and the effluents from anaerobic chambers

From the chart, it is quite evident that the phosphate concentrations were higher in the effluents from the anaerobic chambers than in the Raw Wastewater except in the first week of operation which implies that in that first week, the septic conditions were not yet reached in the anaerobic chambers and therefore the Phosphate Accumulating Organisms which do release phosphate in anaerobic conditions had not colonized the chambers. The eleventh week also depicted equal concentration of phosphate in all the three samples analysed. This is the week that the anaerobic chambers were emptied of the sludge that had accumulated at the bottom.

The reason for having hire concentrations of phosphate in the anaerobic chambers than in the Raw Wastewater is due to the growth of Phosphate Accumulating Organisms (PAO) in the anaerobic chambers. Under anaerobic conditions, PAO assimilate fermentation products (volatile fatty acids) into storage products within the cells while releasing phosphate from stored polyphosphates (Rybicki., 1997). This is actually the starting point for the biological treatment of phosphorus.

Total Phosphorus concentrations in any wastewater is mostly higher than any other form of phosphorus as was witnessed in this study. The study indicated a higher concentration of Total phosphorus as compared to those for phosphates. Given that the wastewater used in this research was a kitchen wastewater, phosphorus was more in the particulate form and therefore, these were reduced in the anaerobic chambers through sedimentation of the particles. Some of the particulate phosphorus were also transformed into soluble phosphorus through biological decomposition resulting to increase in phosphates level as explained above. The Total Phosphorus concentrations measured in the raw wastewater throughout the treatment period ranged between 4.09Mg/L and 8.40Mg/L, with an average of 6.51Mg/L. By passing the Wastewater through the anaerobic chambers, average treatment of 32.42% and 32.58% were achieved in the anaerobic chamber 1 and 2 respectively. The treatment trends are illustrated in Figure 4.10.

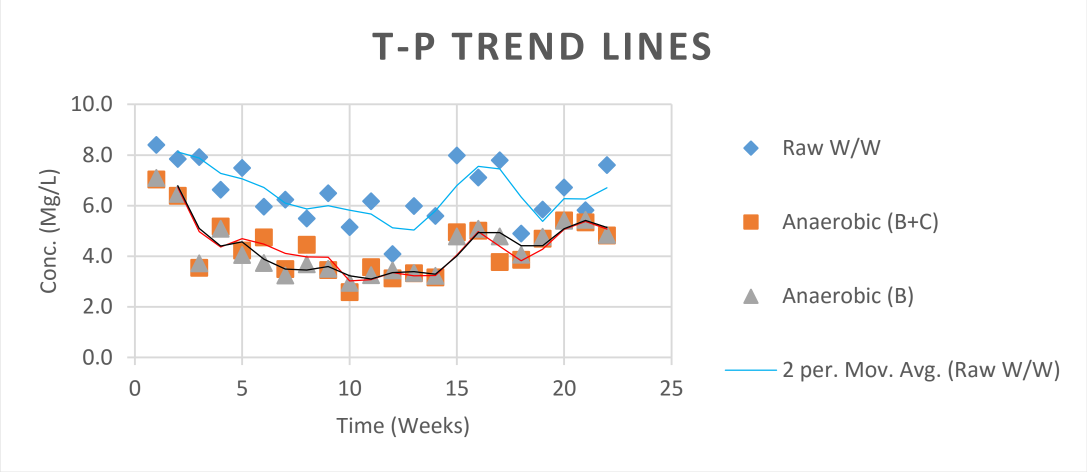

Figure 4.10: Trend lines for T-P concentrations in the Raw Wastewater and the effluents from anaerobic chambers

The chart clearly indicates that the Raw Wastewater had higher concentrations of Total Phosphate throughout the study period than those recorded in the effluents from the anaerobic chambers. It was also worth noticing that towards the end of the study period, the difference in concentration between the Raw Wastewater and the effluents reduced. This could suggest that the system had acquired its run time and therefore needed to be maintained before continuing. It is also visible from the charts that the two anaerobic chambers were recording almost similar readings, with some small differences in some of the weeks in which sampling was done. This shows that the two anaerobic chambers were operating in a similar way.

To further illustrate on the performance of the chambers in terms of reducing the Total Phosphate in the Wastewater, it was necessary to plot the percentage performances of the chambers as from the beginning to the end of the period of study as below as in Figure 4.11.

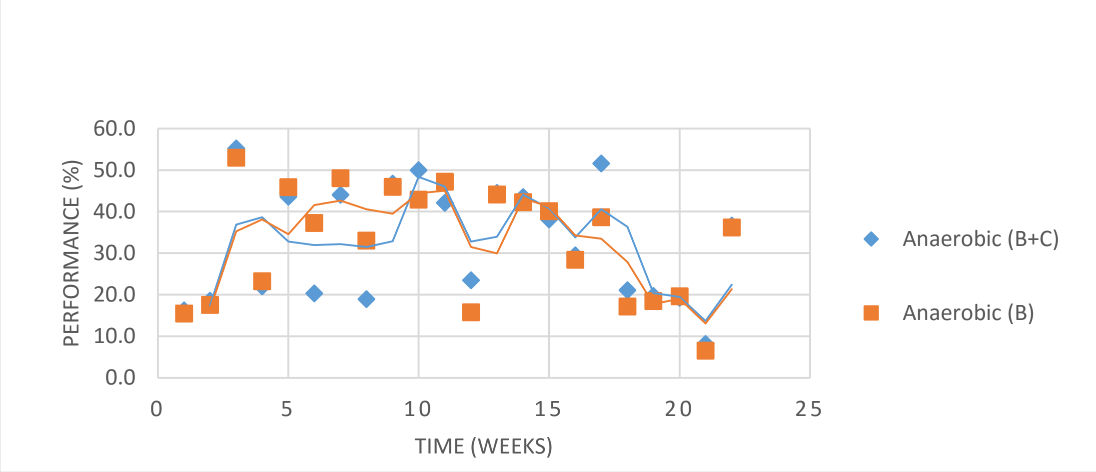

Figure 4.11: An illustration of the performance of the anaerobic chambers in terms of T-P removal

Figure 4.11 illustrates that the two anaerobic chambers were performing well over 10% in removing Total Phosphorus in the Wastewater. The trend lines also show that the treatment started low in the first week, with gradual increment up to its peak in the tenth 10^{th}. Thereafter, there was a gradual decrease in performance to the final week which went below 10%. This shows that the systems took some time to mature for the four weeks. Between the fifth and the fifteenth week, they were at their maximum performance with some few technical hitches interfering with the operations. It can be noted that on the 6^{th}, 8^{th} and 12^{th} weeks, the anaerobic chambers were performing below the expected rates. During this time, a lot of blockages were experienced in the outlet of the chamber. This could have interfered with its performance. After the 15^{th} week, the efficiency started going down since the system was approaching its end of run time.

In the biological removal of phosphorous, the particulate phosphorous in the influent wastewater is incorporated into cell biomass, which is subsequently removed from the process as a result of sludge wasting. The anaerobic chambers were provided with space for settling of sludge at the bottom of the chambers. The sludge was occasionally removed from the chambers together with the attached phosphorus. This is the main process through which the Total Phosphorus was removed from the Wastewater.

4.2.3 Treatment in the slanted filter bed chambers

The filter beds were the main treatment units in both systems. They were expected to provide for treatment through some of the known mechanisms such as mechanical filtration, biological degradation, and adsorption. The beds were filled with treatment media to ensure the mentioned treatment mechanism would occur. A 2.5% gradient was provided to the beds to allow for gravitational flow of wastewater through them.

The filter beds received effluents from the anaerobic chambers as their influents. The effluent from the filter beds were analysed to determine how much of the pollutants coming from the anaerobic chambers were removed in the beds. The quality of the filter bed effluents gave the overall performance of the entire system in relation to the raw wastewater quality. The parameters analysed are presented and discussed in the subsequent subsections.

4.2.3.1 BOD_5 removal in the filter beds

Table 4.5 is a summary of BOD_5 concentrations in the influents into and out of the filter beds. The influents into the beds were from the anaerobic chambers where the wastewater had undergone anaerobic degradation.

Table 4.5: BOD_5 Concentrations in the influents into and effluents out of the filter beds

	Anaerobic Chamber Effluents		**Filter Bed Effluents**		**Reduction rate (%)**
System Type	BOD_5 Range (Mg/l)	Average BOD_5 (Mg/l)	BOD_5 Range (Mg/l)	Average BOD_5 (Mg/l)	BOD_5 Reduction (%)
System 1 (Study system)	162.6 - 294.3	235.1	1.22 - 18.92	6.07	97.61
System 2 (Control system)	144.3 - 310.3	227.3	11.54 - 49.77	21.82	90.63

As much as the anaerobic chambers had reduced the BOD_5 concentrations in the raw wastewater by about 55% as discussed in subsection 4.2.2.1, it was noted that the concentrations were still high in the anaerobic chamber effluents ranging from 235.1 to 227.3Mg/l in both anaerobic chambers for the two systems. Passing the anaerobic chamber effluents through the filter beds showed a tremendous reduction of the BOD_5 concentrations in the wastewater signalling very high efficiency of treatment. The (B+C) filter bed reduced the BOD_5 concentrations from 235.1Mg/L to 6.07Mg/L representing average treatment efficiency of 97.61%. The (B) filter bed on the other hand recorded a 90.63% efficiency in reducing the BOD5 concentration from 227.3Mg/L to 21.82Mg/L. This indicates that the filter bed filled with crushed bricks mixed with charcoal, (system 1) performed better than that filled with crushed bricks alone (system 2)

The treatment trends for the 22 weeks period of the wastewater purification process in the filter beds is illustrate in Figure 4.12.

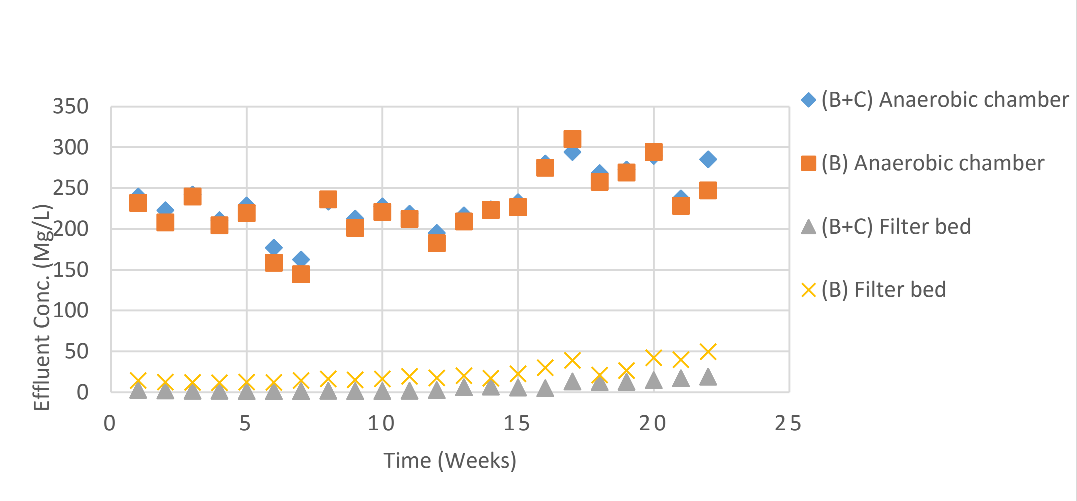

Figure 4.12: BOD5 concentration trends in the anaerobic chambers and Filter beds effluents

Observations from figure 4.12 indicate that the (B+C) filter bed effluent always recorded lower concentrations of BOD_5 as compared to (B) filter bed. This shows that bricks if mixed with charcoal and used as a treatment media in wastewater treatment is superior in removing organics from wastewater when compared to crushed bricks alone. An important point to note is that the two filter beds started by recording slightly higher BOD5 concentrations in their effluents in the first two weeks after which the values went down to a stable condition up to about the 15th week when fluctuations in the values started showing up again.

Amending the crushed bricks with charcoal gave a better result in the filter bed in terms of organic matter (BOD_5) removal since charcoal is highly porous and therefore has a large specific surface area. This enhances the adsorption of the organic matter onto the charcoal surfaces, (Dalahmeh, 2013). The adsorbed organic matter provides food substrate for the bacteria inhabiting the charcoal surfaces. This facilitates biological mineralisation of the sorbed organic matter and oxidation of NH_4+ leading to achievement of relatively high BOD_5 and COD reduction and improved nitrification.

Furthermore, the large specific surface area of the charcoal makes it a good habitat for many bacteria which feed on the organic matter present in the wastewater.

During the first two weeks of operation, there was low BOD_5 removal in the two filter beds since the systems were still maturing up by establishing the necessary micro-organisms for degradation of the organic matter. This happens when biofilms are still forming around the filter materials that to provide feeding material for aerobic bacteria (Dawes, 2006), (Nnaji et al., 2013). After full maturation, the systems performed at high at their high efficiency with peak at weak ten with the highest efficiencies of 99.42% and 95% for the (B+C) and (B) filter beds respectively. The negative trend of performance reduction started at week 12 to the lowest of 93.37% in the 22nd week of operation for the (B+C) Filter bed and 80% for the (B) filter bed. This drop could be attribute to overgrowth of the micro-organisms in the system leading to their death and therefore reducing the system performance (University of Minnesota, 2011). This signalises that the systems were approaching the end of their run time.

A very important observation was at week 17 when the (B+C) filter bed recorded a first reading of over 10Mg/L of BOD_5 concentration in its effluent from the lowest of 1.22 Mg/L and the trend continued with the readings rising for the remaining five weeks of operation. The decision to stop the system was based on this observation.

4.2.3.2 Nitrogen removal in the filter beds

After denitrification processes in the anaerobic chambers, the wastewater was passed through a second process of nitrification in aerobic conditions for effective removal of nitrogen pollutants. The filter beds were designed to provide for aerobic conditions by making them shallow (0.15 M deep) and open for adequate supply of oxygen to the aerobic bacteria (aerobes or nitrobactor) present in the beds. Increased surface area for oxygen supply in the beds was also provided for by using five small trays with wide open tops arranged above each other and field with treatment media material instead of using one big tray with equal capacity. The materials also provided for surfaces on which the aerobic bacteria would reside.

The effluent from the anaerobic chambers were passed through the beds for further treatment and the subsequent nitrogen (NH_4^+-N, NO_2-N, NO_3-N and T-N) concentrations in the final effluents from the filter beds were recorded. Nitrogen concentrations obtained after the analysis of the effluents from both the anaerobic chambers and the filter beds is in Table 4.6.

Table 4.6: Nitrogen Concentrations in the anaerobic chambers and the filter bed effluents

		Anaerobic Chamber Effluents		**Filter Bed Effluents**		**Reduction rate (%)**
System Type	Parameter	Range (Mg/l)	Average (Mg/l)	Range (Mg/l)	Average (Mg/l)	BOD 5 Reduction (%)
System 1 (Study system)	NH_4	0.88 - 1.97	1.6	0.12 - 0.68	0.49	69.54
	NO_2	0.02 - 0.04	0.03	0.05 - 0.10	0.07	-155.99
	NO_3	0.31 - 1.00	0.62	4.31 - 7.51	5.8	-835
	T-N	3.26 - 12.22	7.11	0.82 - 7.33	3.59	49.82
System 2 (Control system)	NH_4	1.14 - 2.04	1.65	0.18 - 0.93	0.59	63.96
	NO_2	0.02 - 0.04	0.03	0.09 - 1.12	0.44	-1289.21
	NO_3	0.08 - 1.07	0.6	10.12 - 52.5	27.55	-4491
	T-N	4.08 - 10.59	7.03	0.82 - 8.15	3.52	50.57

It was observed that out of the four forms of nitrogen pollutants, only two (NH_4^+-N and T-N) were reduced by the filter beds while the remaining ones (NO_2-N and NO_3-N) increased a lot.

Ammonium-Nitrogen was reduced in both filter beds, whereby the (B+C) Filter bed reduced the concentration from 1.60Mg/L to 0.49Mg/L, while the (B) Filter bed reduced the same from 1.65Mg/L to 0.59Mg/L on average. It was also noted the (B+C)

Filter bed had a greater performance of 69.54% Ammonium removal as compared to 63.94% for the (B) Filter bed. The general performance of the two Filter beds in removing the remaining Ammonium-Nitrogen in the effluent from the anaerobic chambers during the period of study is illustrated in the Figure 4.13.

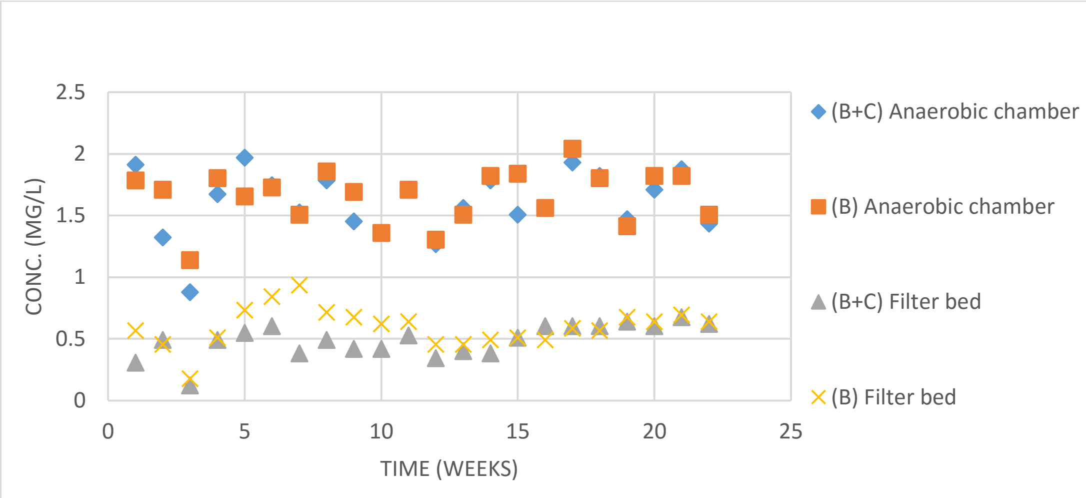

Figure 4.13: Ammonium concentration trend lines in the anaerobic chambers and Filter beds effluents

The effluent from anaerobic chambers had higher concentrations of ammonium as compared to effluents from the filter beds. The (B+C) filter bed gave slightly lower concentrations of ammonium in its effluent than that from the (B) filter bed, especially during the first 15 weeks of operation. This is a demonstration that the (B+C) filter bed was performing better than the (B) filter bed. The difference in performance between the two filter beds is demonstrated by performance efficiency plot for all the period of study in Figure 4.14.

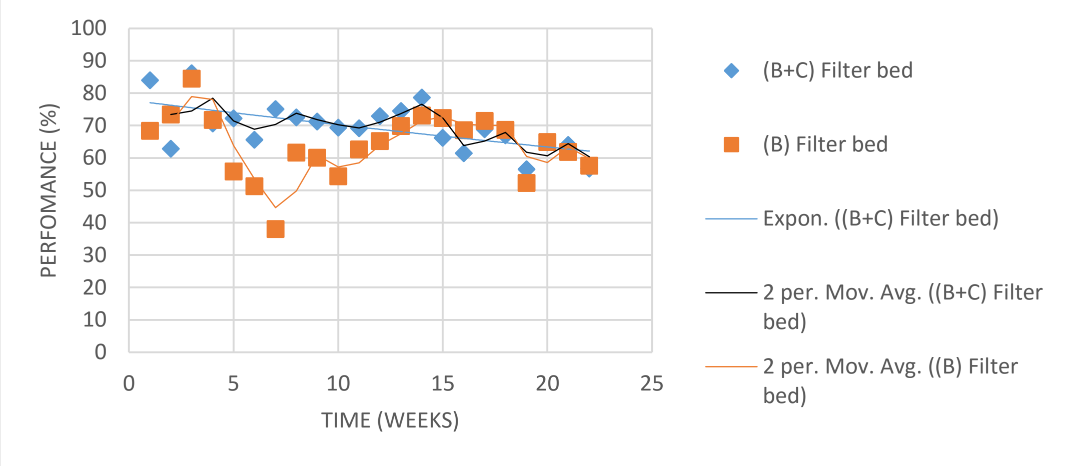

Figure 4.14: An illustration of the performance of the Filter beds in terms of NH4 removal

Figure 4.14 shows that there was a difference in performance between the two filter beds, with (B+C) filter bed performing better than (B) filter bed in reducing the ammonium concentrations. In the first four weeks of operation the two systems were maturing by accumulating the necessary microorganism required for transforming the ammonium to nitrites and nitrates (nitrification processes).

Ammonium reduction in the filter beds as illustrated in Figure 4.16 was because of **nitrification processes** which were taking place in the beds. Nitrification is the process through which the Ammonium-Nitrogen in the wastewater is converted into Nitrites and finally Nitrates through aerobic reaction by the aerobic bacteria. For this process to take place adequately, oxygen supply should be sufficient and if possible, aeration of the beds should be done by bubbling oxygen in. As much as the beds were made shallow to provide for adequate supply of oxygen, the bottom parts of the beds were saturated, and no aerobic processes were taking place in the lower parts of the filter beds, and this is the reason why maximum removal of Ammonium was not achieved. This could only be achieved by introducing diffusers for bubbling of oxygen into the filter beds. The difference in performance of the two filter beds is due to incorporation of charcoal in the (B+C) filter bed. Charcoal made from maize cobs is porous giving it large surface area for its adsorption properties. This provided the filter bed with bricks mixed with charcoal advantage of adsorbing more ammonium ions on the charcoal surfaces over the filter bed with bricks a lone as the filter materials (Gai et al., 2014; Foereid, 2015; Hollister et al., 2013).

As a result of nitrification in the filter beds, Nitrite and Nitrate concentrations increased by more than 100% in the effluents from the beds. During nitrification process, Ammonium is first oxidized to Nitrite and eventually to Nitrate since Nitrite is unstable compound. Because of its instability, Nitrites concentrations were less in the effluents from the two filter beds as compared to Nitrates since much of it were converted to nitrates through oxidation. A point to note, is the difference in Nitrite concentrations in the two filter bed effluents. The (B+C) filter bed increased the Nitrite levels in the Wastewater from 0.03Mg/L to 0.07Mg/L, which translated to 156% increase in concentration in average. Likewise, the (B) filter bed increased the Nitrite concentration in the same wastewater from 0.03Mg/L to 0.40Mg/L, translating to 1289.21% average increase. The increase of the Nitrite concentration in the bricks filter bed was much higher compared to what was achieved in the (B+C) filter bed.

A similar observation was also made with Nitrate whereby the increase was from 0.62Mg/L to 5.80Mg/L, translating to 910.96% in the (B+C) filter bed, while the (B) filter bed increased the concentrations by 5179.46%, from 0.6Mg/L to 27.55Mg/L in average.

The main cause of very high concentrations of Nitrites and Nitrates in the effluents from the filter beds was the nitrification processes that were taking place in the beds. The ammonium which was coming in from the anaerobic chambers was acted upon by the aerobic bacteria that occupied the beds. For this to happen, a sufficient supply of oxygen was enabled by providing large surface area of the bed troughs. Key to mention is the nature of the filter media in the filter beds which brought about the difference in the levels of concentration of the two pollutants in the two filter bed effluents. In chapter two, it is stated that charcoal is good in adsorption of pollutants such as ammonium, phosphates, and organics (Dalahmeh, 2013; Foereid., 2015). The organic wastes and the ammonium are necessary for formation of Nitrites and Nitrates in the sense that the organics are converted to ammonium in a denitrification process thereafter the ammonium is converted to Nitrites and finally nitrates in a nitrification process. Both processes were present in the filter beds. The difference in levels here comes in where charcoal is mixed with bricks increases the adsorption property of the filter material. In this case, the much of the organic waste (BOD_5) and ammonium which came from the anaerobic chambers were adsorbed on the charcoal surfaces in the (B+C) filter bed making them unavailable for conversion to both Nitrites and Nitrates. This could be one reason why the two (NO_2 and NO_3) were less in the (B+C) filter bed effluent as compared to that from bricks. Another reason for the difference in the level of increase in both Nitrites and Nitrates in the Filter bed effluents was as earlier discussed in chapter 2 that charcoal has high affinity for oxygen. Because of this, the charcoal in the filter beds were adsorbing some of the oxygen which could be used for nitrification processes on its surfaces. This limited the processes leading to less Nitrite and Nitrate formation in the (B+C) filter bed as compared to (B) filter bed which did not have the capacity to hold some oxygen hence most of it were available for denitrification process in the filter bed. This the reason why there was more of these two pollutants in the (B) filter bed effluent than in that from (B+C) filter bed.

The study was aimed at treating wastewater of all the pollutants related to greywater from kitchen. However, this was not the case for Nitrites and Nitrates. Instead, the levels of these two increased to very dangerous levels. The danger of nitrates in drinking water arises when it is present at high levels and may cause illnesses in infants and other vulnerable persons (Hyngstrom et al., 2011).

There are four conditions which must be met for denitrification of NO3 to N_2 and N_2O gases to effectively take place (Wu et al., 2010). These conditions are:

- Oxidation of NH_4^+-N to NO_3^--N (nitrification);
- Subsequent anaerobic conditions.
- Presence of denitrifying bacteria; and
- An adequate carbon (energy) source for the denitrifying bacteria present in the anaerobic zone.

For the case of this study there was no reduced conditions immediately following the nitrification process in the filter beds and at the same time, there was no suitable carbon source which could assist in denitrification of NO_3 to N_2 which could be released in the environment as a gas.

Nitrate is also an essential nutrient for plants growth and therefore, introducing some forms of plants on the filter bed could have helped in reducing the concentrations in the filter bed effluents. However, the plants would contribute to water loss through evapotranspiration, and this led to exclusion.

Also reduced in the filter beds was the Total Nitrogen concentrations. From a concentration of 7.11 Mg/l in the effluents from the anaerobic chamber, the (B+C) filter bed reduced this to a concentration of 3.59 Mg/l giving a percentage reduction of 49.82% while the bricks filter bed reduced the concentrations by 50.57% from a concentration of 7.04Mg/l to 3.52Mg/l on average. The major constituent of Total Nitrogen is the organic nitrogen whose quantity was reduced in the filter beds through mechanical filtration, adsorption, and biological degradation. It is again evident that the filter bed with bricks amended with charcoal was a little bit superior in the removal of Total Nitrogen as compared to that of bricks due to improved adsorption property of the bricks and charcoal filter bed.

4.2.3.3 Phosphorus removal in the filter beds

Phosphorus ions are negatively charged and are capable of being strongly bound to hydrous oxides of the filter materials through a process known as adsorption, after which they are held on the surfaces (Loomis, 1999; Dawes, 2006; Hyngstrom et al.,2011). This process is also called chemical precipitation of phosphorus (Rybicki., 1997). This was witnessed in this study as much of the phosphorus present in the wastewater was removed in the filter beds as compared to the anaerobic chambers. This can be illustrated by a show of a Table 4.7 indicating phosphorus concentrations in the wastewater coming into the filter beds from the anaerobic chambers and that coming out of the beds.

Table 4.7: Phosphorus Concentrations in the anaerobic chambers and the filter bed effluents

		Anaerobic Chamber Effluents		**Filter Bed Effluents**		**Reduction rate (%)**
System Type	Parameter	Range (Mg/l)	Average (Mg/l)	Range (Mg/l)	Average (Mg/l)	Reduction (%)
System 1 (Study system)	PO_4	2.04 – 4.4	3.12	0.16 - 0.33	0.23	92.13
	T-P	2.58 – 7.03	4.37	0.45 – 1.9	0.63	85.39
System 2 (Control system)	PO_4	1.92 – 4.68	3.23	0.13 - 0.71	0.28	90.6
	T-P	2.94 – 7.1	4.36	0.5 – 1.62	1.00	75.94

By passing the anaerobic chamber effluents through the filter beds, a substantial decrease in Phosphates was achieved as presented in table 4.7. The (B+C) filter bed reduced the Phosphate concentration by about 92% from an average concentration of 3.11Mg/l to 0.23Mg/l while the bricks filter bed reduced the concentrations by about 90% from an average of 3.23Mg/l to 0.28Mg/l. This again shows that the (B+C) filter bed gave a slightly higher performance in removing Phosphates than bricks filter bed. Amending the bricks with charcoal improved the efficiency of the filter bed due to the large specific surface area of the charcoal which enhanced adsorption capacity of removing of Phosphates (Dalahmeh, 2013).

The (B+C) filter bed reduced the Total Phosphorus by 85.39% from an average concentration of 4.4Mg/l in the effluent from its anaerobic chamber to a concentration of 0.63Mg/l in the (B+C) filter bed effluent. The bricks filter bed on the other side reduced the Total Phosphate concentrations from 4.36Mg/l in its anaerobic chamber effluent that was feeding the beds to an average concentration of 1.0Mg/l in the bricks filter bed effluent, giving an average level of performance of 75.95%. From the percentages, it can be observed that the (B+C) filter bed performed better than the one for bricks in terms of Total phosphorus removal. This observation was made in the entire period of study as illustrated in Figure 4.15.

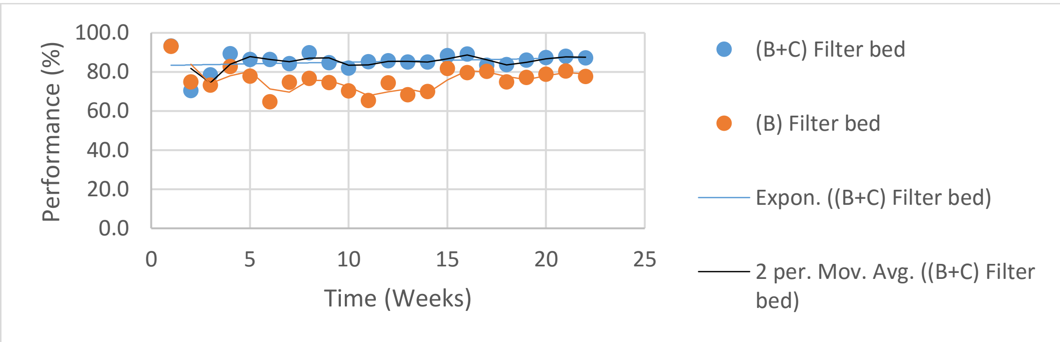

Figure 4.15: Performance efficiency of the filter beds in removing Total Phosphorus from wastewater

In Figure 4.16, the (B+C) filter bed gave better performance in almost all the tests carried out in the whole study period. This better performance could be attributed to by the large specific surface area of the charcoal, enhancing adsorption of soluble phosphorus and the capacity for biological mineralisation of organic matter and removal of particulate phosphorus (Dalahmeh, 2013).

4.2.4 Overall performances of the two systems

Generally, the two systems that were set up for study performed well in treating the kitchen wastewater of the major pollutants associated with the kitchen waste except Nitrites and Nitrates whose concentrations were much higher in the final treated effluents than in the original wastewater. However, the system which was being studied that had a mixture of crushed bricks with charcoal amendment as the filter material proved to be performing better than the control system that had crushed bricks only in treating the wastewater. The general performance of the two systems in terms of treating the wastewater of individual pollutant is discussed in the subsequent subsections with figures showing the differences in treatment efficiencies for the two systems.

4.2.4.1 BOD_5 Removal

In overall, the two systems exhibited good performance in treating the wastewater of the organics by providing average performance efficiency of over 95% in both cases for the 22 weeks of operation. The (B+C) System indicated a better performance by averagely removing 97.61% of the BOD_5 present in the raw wastewater as compared to the (B) System which removed 95.69%. The treatment was a two-step as shown in Figure 4.16.

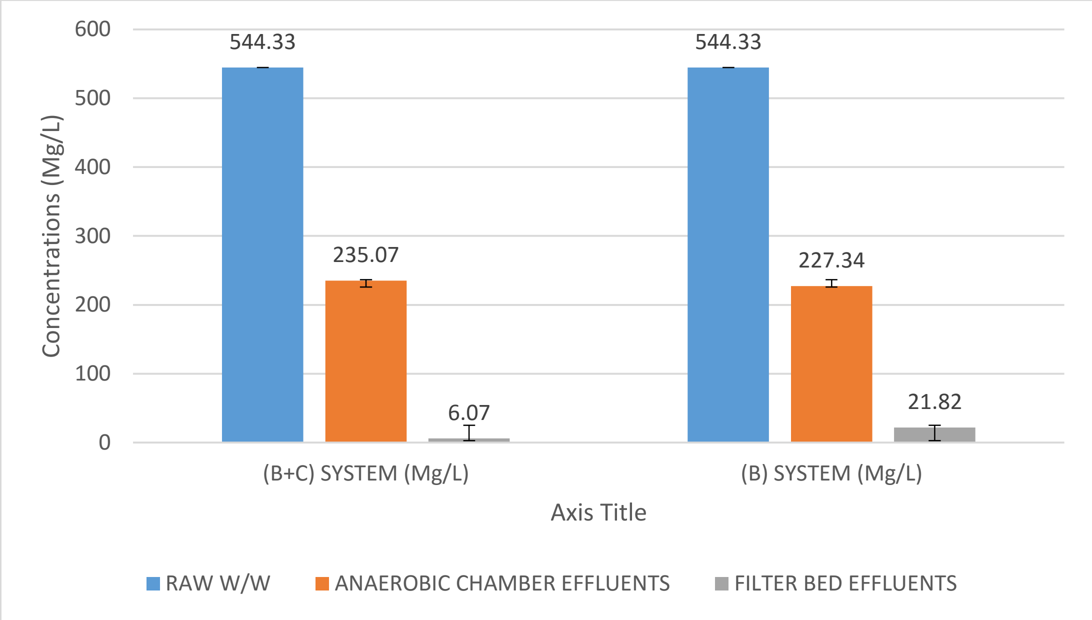

Figure 4.16: BOD5 Concentrations in different components of the treatment systems

The treatment process started in the anaerobic chambers which removed about 50% of the organics in both cases while the filter beds removed the rests. The figure shows that the raw wastewater had very high organic content of 544.33Mg/l and after passing it through the two wastewater treatment systems that were set-up, the concentrations reduced drastically in both cases to a final level of 6.07Mg/l for the (B+C) System and 21.82Mg/l for the (B) Systems on average.

All most all the guidelines and standards for wastewater reuse are mainly for irrigation purposes because irrigation is the highest consumer of water in any country and therefore given the priority in reuse of wastewater. The most restrictive standards criteria require a BOD_5 of less than 10 Mg/l. However, other countries do not have very stringent guidelines and can allow higher concentrations of the different parameters or do not include them at all (Jefferson et a.l, 2004). For instance, the (WHO, 2006) guidelines for reusing greywater for irrigating ornamentals, fruit trees and fodder crops requires a BOD_5 of less than 240Mg/l, for irrigating vegetables likely to be eaten uncooked BOD_5 of less than 20Mg/l and for toilet flushing requires BOD_5 of less than 10Mg/l.

Based on these guidelines and standards, the effluents from (B+C) System is suitable for toilet flushing as well as surface or spray irrigation of any food crop, including crops eaten raw wile that from the (B) System can be used for surface irrigation of Orchards, Vineyards and non-food crops (Ruiz-Palacios and Mara, 2000). While organics in effluent when applied at an appropriate rate can revitalise soil fertility, continued overloading of organic matter may physically clog soil pores and favour anaerobic microbiological populations in the soil. This is why the levels must be controlled.

The (B+C) system met the stringent standards of the American National Standards (ANSI/NSF 350 and 350-1) which requires that implementation of residential and commercial on-site and greywater treatment systems should produce effluents with test average of BOD_5 of 10Mg/l and maximum single sample of BOD_5 of 25Mg/l for both Class R and Class C as in the United States Environmental Protection Agency, (US EPA, 2012).

4.2.4.2 Nitrogen Removal

This study revealed that about 50% of ammonium was removed from the wastewater by the two systems. To be specific, the (B+C) system removed on average 69.55% of the 1.26Mg/l ammonium concentration which was present in the raw wastewater to yield an effluent with average concentration of 0.49Mg/l. The (B) System on the other hand produced an effluent with average concentration of 0.59Mg/l ammonium from the initial concentration of 1.26Mg/l in the raw wastewater. This translates to Ammonium removal of 50.74%. Comparing the two systems, it was noted that the (B+C) System was performing slightly better than the (B) System. This was due to improved adsorption capacity of the (B+C) system brought about by adding charcoal in the filter media. The anaerobic chambers provided for anaerobic conditions for conversion of organic nitrogen to ammonium and that is why the ammonium concentrations were higher in the anaerobic chamber effluent than in the raw wastewater. This scenario is demonstrated in Figure 4.17.

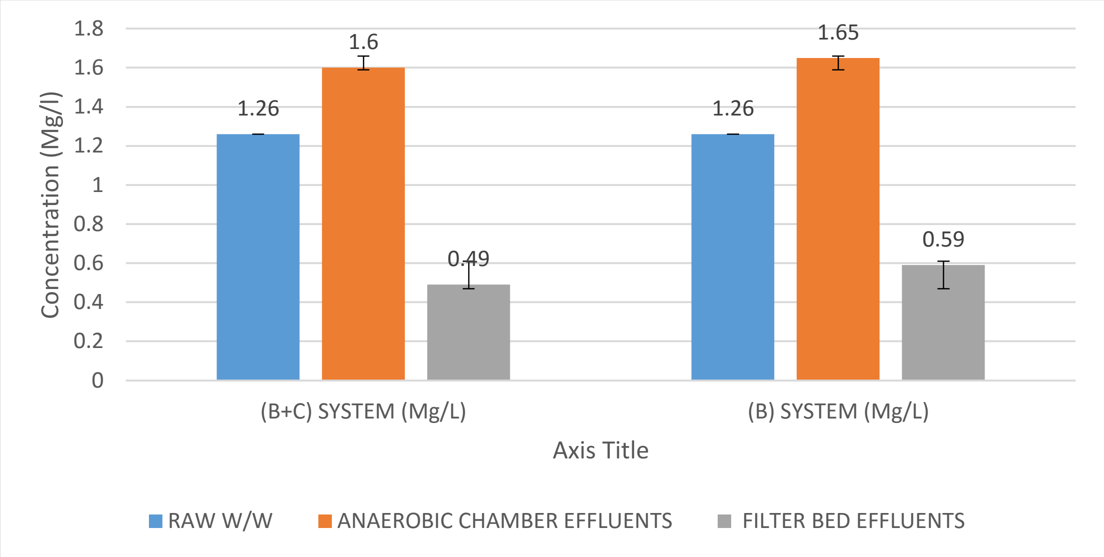

Figure 4.17: Ammonium Concentrations in different components of the treatment systems

Figure 4.17 illustrates that a lot of ammonium were lost in the filter bed. However, ammonium levels increased in the anaerobic chambers due to anaerobic conversion of organic nitrogen present in the raw wastewater into ammonium ions.

In removing Total-Nitrogen, the two systems performed much better than they did in removing ammonium. This was exhibited by over 80% removal of the Total Nitrogen concentrations present in the raw wastewater. The (B+C) System removed 88.7% of the Total Nitrogen initially available in the raw wastewater by reducing the concentrations from an average of 32.65Mg/l in the raw wastewater to 3.6Mg/l in the (B+C) filter bed effluents. The Brick System also performed more less the same as the

(B+C) System by removing 89% of the Total Nitrogen from the raw wastewater that had initial Total Nitrogen concentrations of 32.65Mg/l to yield an effluent with 3.52Mg/l Total Nitrogen concentration. There was no significant difference in performance between the two systems in removing Total-Nitrogen since much of it was removed in the anaerobic chambers which were similar in their design and operation as illustrated in Figure 4.18.

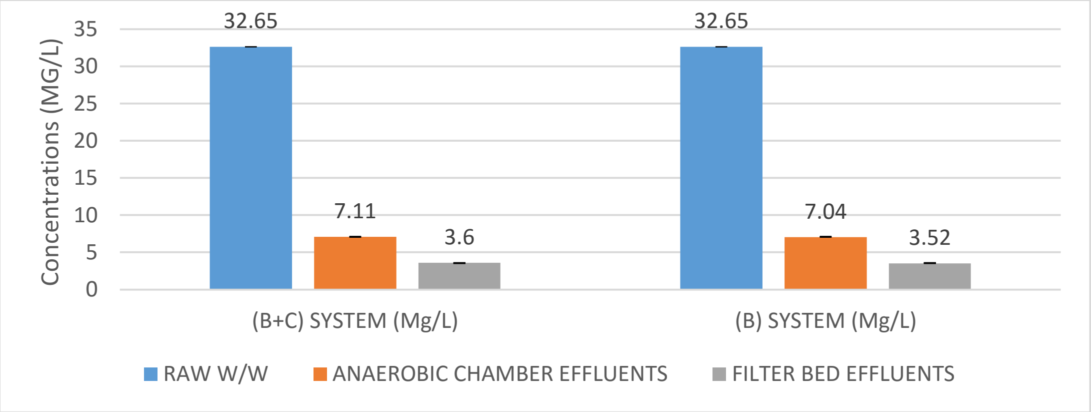

Figure 4.18: Total Nitrogen Concentrations in different components of the treatment systems

Unlike Ammonium, much of the Total nitrogen were removed in the anaerobic chambers as demonstrated in Figure 4.19. The anaerobic chambers removed about 77% of the Total nitrogen available in the raw wastewater after which the filter beds removed the remaining portion. This is because much of the Total Nitrogen were in solid form and settled at the bottom of the anaerobic chambers and later removed as sludge.

For the case of Nitrites and Nitrates, there was a slight decrease in the anaerobic chambers as compared to what was in the raw wastewater. However, there was massive increase in in the concentrations in the final effluents from the filter beds of the two systems as illustrated in Figures 4.19 and 4.20. The increase was due to the nitrification processes that prevailed in the filter beds that converted ammonium ions into nitrites and finally nitrates.

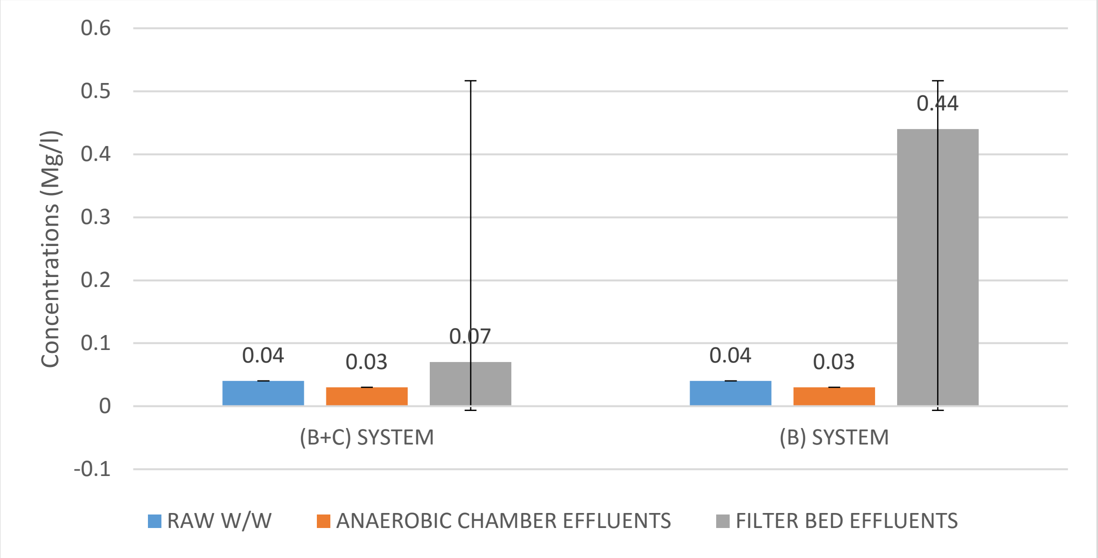

Figure 4.19: Nitrite Concentrations in different components of the treatment systems

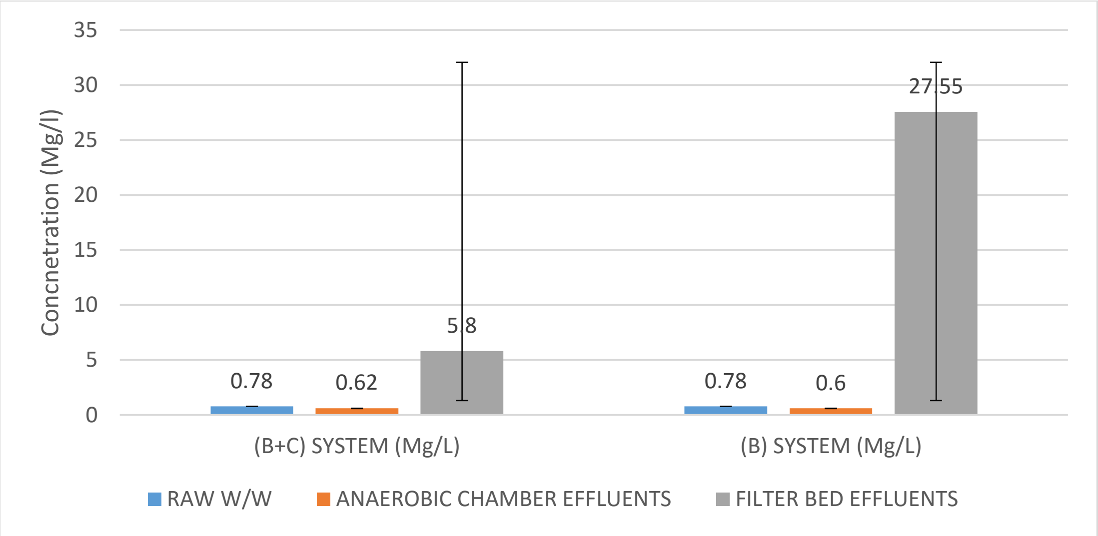

Figure 4.20: Nitrate Concentrations in different components of the treatment systems

Nitrite formation is a transition process during the conversion of Ammonium to Nitrates and that's why Nitrite concentrations were much less than Nitrates. In short, Nitrite is a very unstable compound and therefore oxidises easily to Nitrates. Removal of nitrates from wastewater in the two systems required additional anaerobic unit and some carbon material as a source of energy for denitrification of Nitrates to Nitrogen gas which would have been lost into the air.

Nitrogen is important in helping plants with rapid growth, increasing seed and fruit production, and improving the quality of leaf and forage crops. Therefore, there is added value in reusing effluents from the two systems for irrigation purposes. However, for agricultural reuse Nitrogen may be present in wastewater at concentrations ranging between 10 to 50 mg/L in a variety of chemical forms: organic, ammonia, nitrate, and nitrite. Any excess nitrogen in irrigation water may be carried through the soil to the water table. Elevated nitrogen levels may render groundwater unsuitable for stock and domestic water supplies. Nitrate is a health risk to humans at more than 10 mg/l and animals at more than 100 mg/l (US EPA, 2012).

4.2.4.3 Phosphorus Removal

The removal of phosphorus from the wastewater by the two systems was quite effective, with average removal efficiency of both Phosphates and Total-Phosphorus being over 80% for both systems. Much of the Phosphorus were removed in the filter beds in both cases.

In treating the wastewater of Phosphates, the systems employed two methods; biological treatment method as observed in the increase of Phosphate concentrations in the anaerobic chambers due to Phosphate release from stored polyphosphates within the Phosphate Accumulating Organisms, under anaerobic conditions. This scenario is illustrated in Figure 4.21 which shows that anaerobic chamber effluents had higher Phosphate concentrates than the raw wastewater.

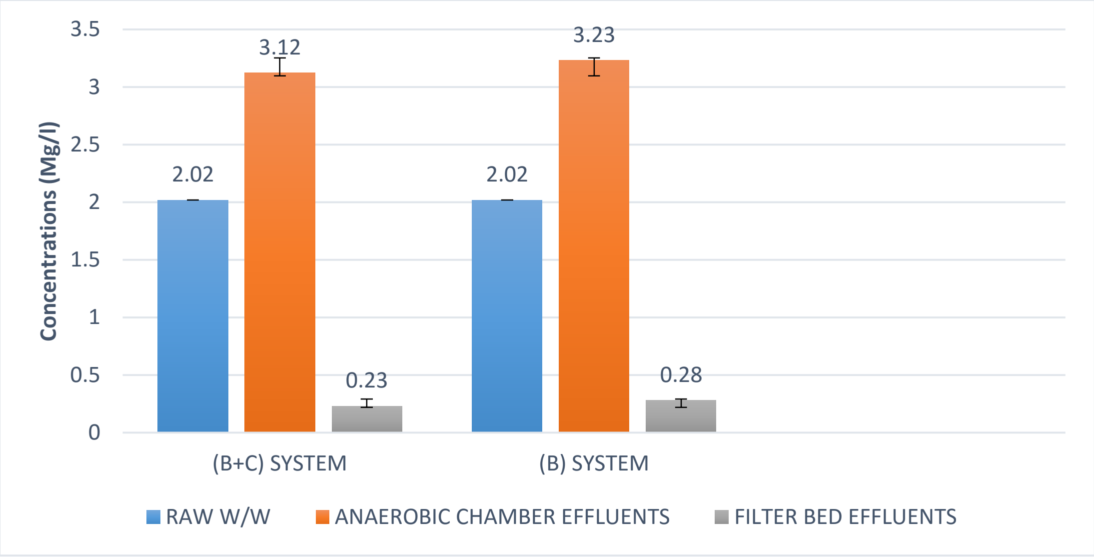

Figure 4.21: Phosphate Concentrations in different components of the treatment systems

Passing the anaerobic chamber effluents through the filter beds enabled production of energy through oxidation of the storage products and polyphosphate storage units within the Bacteria cells increased. This provided for a room for incorporating orthophosphate soluble in wastewater into polyphosphates in the PAO cells which were filtered out with the biomass in the filter beds. This accounts for the full process of biological Phosphate removal from the wastewater. The other method through which phosphates were removed from the wastewater was through adsorption in the filter beds. Phosphate ions are negatively charged and were getting adsorbed onto the filter material surfaces as the wastewater was passing through the filter beds. The charcoal in the (B+C) system improved the adsorption property of the system leading to more Phosphates adsorbed in the (B+C) filter bed than in the (B) filter bed as evident in the concentration difference in the filter bed effluents with (B+C) system giving 87.16% Phosphate removal whereas (B) system gave 83.83% Phosphate removal. The two methods combined led to effective phosphate reduction in the wastewater by the two systems by reducing the Phosphate concentrations from 2.02Mg/l in the raw wastewater to 0.23Mg/l and 0.28Mg/l in the (B+C) system and (B) system effluents respectively.

Total-Phosphate removal was also quite effective whereby the anaerobic chambers removed about 30% of the total Phosphate present in the raw wastewater while the filter beds removed the remaining, leading to total removal of 90.3% for (B+C) system and 84.18% for (B) system. The concentration levels of Total-Phosphate at every stage of treatment is as shown in Figure 4.22.

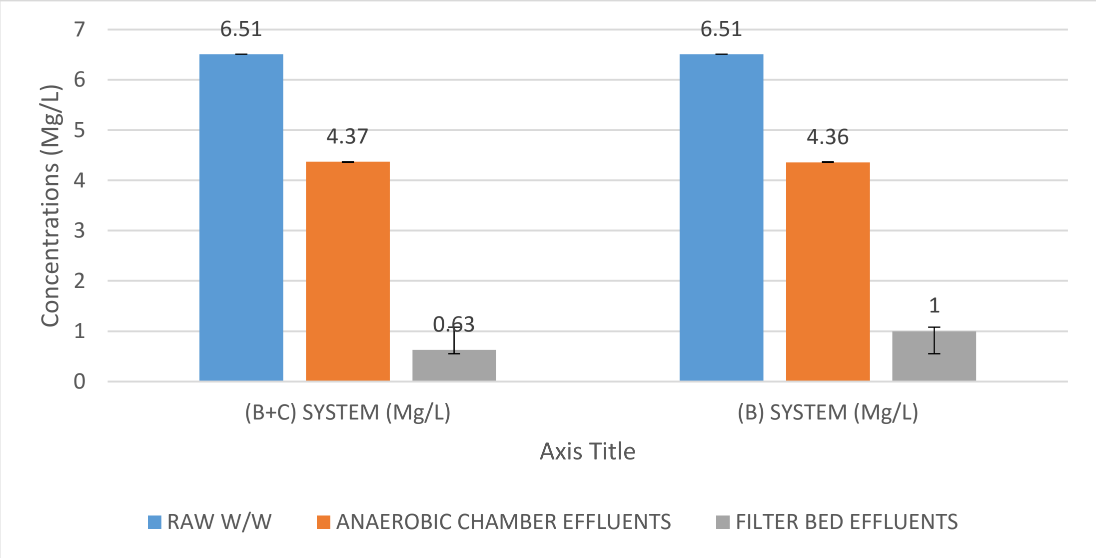

Figure 4.22: Total-Phosphorus Concentrations in different components of the treatment systems

Figure 4.22 reveals that there was significant Total-Phosphorus removal from the wastewater by the two systems. The (B+C) System performed overwhelmingly well by reducing the Total-Phosphorus concentrations in the wastewater from a very high average of 6.51Mg/l to 0.63Mg/l. The (B) system also performed well by reducing the same from 6.51Mg/l to1.0Mg/l on average. The two means indicate a significant difference in the operation of the two systems. . It can be concluded that the (B+C) System performed better in removing the Total-Phosphorus than the (B) System. However, this had to be proved by conducting statistical analysis to determine if there is any significant difference in the means.

The success of removing Phosphorus biologically depends upon the amount of organic material, expressed as either BOD or COD, and total phosphorus in the influent wastewater entering the anaerobic selector. An adequate amount of organic material must be available to support phosphorus-accumulating organisms (PAO). Studies of operations have shown that a BOD/phosphorus ratio of at least 20:1 or a COD/phosphorus ratio of at least 45:1 is needed for enhanced biological Phosphorus removal (Wisconsin Department of Natural Resources, 2015). Based on the above facts, the higher removal efficiency of Phosphorus by the two systems could be attributed to the high levels of BOD5 witnessed from the wastewater water.

Phosphorus is an essential element for plant life, but when there is too much of it in water, it can speed up eutrophication (a reduction in dissolved oxygen in water bodies caused by an increase of mineral and organic nutrients) of rivers and lakes and that's why it is important to be removed from in wastewater to be discharged in water bodies.

4.3 Statistical Analysis

One of the objectives in this research was to find out which of the two system set-ups would provide better result in treating domestic wastewater for the purpose of reuse in non-portable household water uses. To determine this, a statistical analysis was done to find out if there was any difference in performance between the two systems and if the difference was significant.

A two-sample *t*-test was used in this case to test if the improvement made on the crushed bricks by amending it with charcoal brought about any difference in the means of the two data sets collected from the effluents from the two filter beds. The control system had crushed bricks alone as the filter material while the other system had crushed bricks amended with charcoal whose effect was being determined. The test was therefore done to determine if the two-population means were equal.

To conduct this, R software was used. R is a language and environment for statistical computing and graphics. Biological Oxygen Demand (BOD_5), Total-Nitrogen (T-N) and Total Phosphorus (T-P) were picked for this analysis since they are the main pollutants in domestic wastewater (grey water) while the other pollutants (PO_4, NO_3 and NO_2) are constituents of Total Phosphorus and Total Nitrogen. Given that the two systems provided two different sets of data per parameter analysed. The data was fed into the software and two-sample t-tests conducted on the BOD5, Total-Nitrogen and Total Phosphorus data sets to determine if there was any significant improvement in removing the three pollutants by amending crushed bricks with charcoal and used to purify wastewater if compared to when crushed bricks is used alone.

4.3.1 T-test for Biological Oxygen Demand

Figure 4.23 presents data extracted from R indicating two sample t-test output for BOD_5.

```
> attach(mydata)
> mydata
   EFLUENT.BC EFLUENT.B
1       3.120    14.270
2       2.274    12.276
3       1.603    11.813
4       1.738    11.540
5       1.223    12.260
6       1.367    11.820
7       1.243    14.230
8       1.653    16.253
9       1.236    15.236
10      1.325    16.256
11      1.743    19.340
12      2.677    17.260
13      5.960    20.140
14      6.710    16.941
15      5.850    22.400
16      4.960    29.880
17     13.020    39.050
18     12.450    20.870
19     12.940    26.300
20     14.620    42.300
21     16.890    39.820
22     18.920    49.770
> t.test(EFLUENT.BC,EFLUENT.B)

        Welch Two Sample t-test

data:  EFLUENT.BC and EFLUENT.B
t = -5.8152, df = 31.554, p-value = 1.95e-06
alternative hypothesis: true difference in means is not equal to 0
95 percent confidence interval:
 -21.27016 -10.23011
sample estimates:
mean of x mean of y
 6.069182 21.819318

> |
```

Figure 4.23: T-test data analysis for BOD5 extracted from R

The p-value for the two-sample t-test is 1.95×10^{-6} which was much lower than the 0.05 level of significance. We can therefore be confident at 95% that there is a significant difference between the two means of 6.06 and 21.82 and confidently reject the null hypothesis. It therefore implies of the two systems; the charcoal amendment improved the quality of bricks in terms of cleaning the wastewater of the biodegradable organics.

4.3.2 Total Nitrogen

The extracted data for the analysis is presented in Figure 4.24

```
> mydata1=read.csv(file="C:/Users/GEORGE ALOLO/Desktop/TNANALYS.csv",head=T,sep=",")
> attach(mydata1)
> mydata1
   EFLUENT.BC EFLUENT.B
1      2.4472    3.2618
2      3.2618    2.4472
3      3.2618    1.6327
4      4.8909    3.2618
5      4.0763    3.2618
6      4.8909    4.0763
7      3.2618    3.2618
8      4.8909    4.8909
9      3.2618    2.4472
10     0.8181    0.8181
11     0.8181    0.8181
12     1.6327    0.8181
13     2.4472    0.8181
14     1.6327    2.4472
15     2.4472    0.8181
16     0.8181    0.8181
17     5.7054    8.1491
18     4.0763    6.5200
19     4.8909    7.3345
20     6.5200    5.7054
21     7.3345    7.3345
22     5.7054    6.5200
> t.test(EFLUENT.BC,EFLUENT.B)

        Welch Two Sample t-test

data:  EFLUENT.BC and EFLUENT.B
t = 0.11243, df = 39.283, p-value = 0.9111
alternative hypothesis: true difference in means is not equal to 0
95 percent confidence interval:
 -1.257922  1.406031
sample estimates:
mean of x mean of y
 3.595000  3.520945
```

Figure 4.24: T-test data analysis for Total-Nitrogen extracted from R

The p-value for the two-sample t-test in this case was 0.9111 and was above 0.05 significant level at 95% confidence level. This means that the null hypothesis is true that the two means are the same. We therefore don’t have enough evidence that the charcoal added to the crushed bricks improved the performance of the filter materials in terms of Total-Nitrogen removal. This means that the two systems were performing similarly in removing the Total Nitrogen from the wastewater. The findings in this section conforms to the previous findings in subsection 4.2.3.2 which indicated a very small difference of less than 1% between the two systems in treating the wastewater of the Total Nitrogen. This is because the major portion of the Total Nitrogen were removed in the anaerobic chambers which were very similar in their design, nature, and conditions of operation. The filter beds which had some differences were left with very small quantities of Total Nitrogen to be removed from the wastewater which could not bring any significant difference in the results.

4.3.3 Total Phosphorus

```
> mydata1=read.csv(file="C:/Users/GEORGE ALOLO/Desktop/TPANALYSIS.csv",head=T,sep=",")
> attach(mydata1)
> mydata1
   EFLUENT.BC EFLUENT.B
1      0.4730    0.4973
2      1.8933    1.6270
3      0.7636    0.9976
4      0.5537    0.8765
5      0.5780    0.9007
6      0.6506    1.3204
7      0.5537    0.8200
8      0.4569    0.8604
9      0.5295    0.8927
10     0.4650    0.8765
11     0.5295    1.1267
12     0.4488    0.8846
13     0.4973    1.0621
14     0.4730    0.9734
15     0.5780    0.8765
16     0.5457    1.0379
17     0.6264    0.9411
18     0.6264    1.0218
19     0.6587    1.0863
20     0.6829    1.1428
21     0.6425    1.0621
22     0.6183    1.0863
> t.test(EFLUENT.BC,EFLUENT.B)

        Welch Two Sample t-test

data:  EFLUENT.BC and EFLUENT.B
t = -4.7741, df = 38.19, p-value = 2.655e-05
alternative hypothesis: true difference in means is not equal to 0
95 percent confidence interval:
 -0.5259544 -0.2127638
sample estimates:
mean of x mean of y
0.6293091 0.9986682
```

Figure 4.25: T-test data analysis for Total-Phosphorus extracted from R

The test produced a p-value 2.655×10^{-5} which was much lower than the 0.05 significant level. We can therefore be confident at 95% that there is a significant difference between the two means and confidently reject the null hypothesis and accept the alternative hypothesis that the two means a significantly different. We therefore have evidence that charcoal plays a role in removing Total-Phosphorus when mixed with crushed bricks as the treatment media as initially depicted by the difference in percentages of performance of the two filter beds.

CHAPTER 5: CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The main aim of this research was to contribute to attempts made in providing solutions to water scarcity problems by treating household grey water using locally available materials and reusing the treated water as an alternative/additional source of water for non-portable uses within the household in rural and low-income urban communities in Kenya. Two systems were set up with difference in the filter bed materials. The reason was to determine which filter material would give better treatment efficiency in treating the wastewater to a level recommended for reuse in the non-portable household water uses.

The following conclusions were drawn from the study:

a. Efficiency of wastewater treatment in filter beds depend on the type of filter materials used in the beds. The control and study system filter beds removed substantial quantities of biodegradable organics in the form of BOD_5 by 90% and 97% respectively. Other pollutants that were substantially reduced by the filter beds were Ammonium at 65%, Total Nitrogen at 50%, Phosphates at 91% and Total Phosphorus at 80% on average. From this analysis, it was noted that amending the crushed bricks with charcoal improved the performance of the study system as compared to the control in removing most of the parameters measured. The final treated water from the study system met the stringent standards of the American National Standards (ANSI/NSF 350 and 350-1) which requires that implementation of residential and commercial on-site and greywater treatment systems should produce effluents with test average of BOD_5 of 10Mg/l and maximum single sample of BOD_5 of 25Mg/l for both Class

R and Class C as in the United States Environmental Protection Agency, (US EPA, 2012).

b. Due to the aerobic conditions prevailing in the filter beds, the Nitrates and Nitrites concentration increased in the filter bed due to nitrification of Ammonium within the beds.

c. The two systems performed well up to the 17th week of operation from which the efficiencies started declining. This was an indication that the run time for the filter materials used in the study would range between 15 to 17 weeks before they are cleaned and dried up for reuse.

5.2 Recommendations

This study appreciates advances made in developing on-site wastewater treatment and reuse systems and recommends the following for further studies:

a. Conduct similar study with a more homogeneous greywater obtained from the different sources including bathtubs, hand washing sinks, laundry, and kitchen sinks to reduce on the high biodegradable organics associated with kitchen water when used alone.

b. Include the third step of anaerobic process for denitrification of Nitrites and Nitrates to Nitrogen gas.

c. Explore other modifications or amendment materials as alternative for charcoal or in combined form for better performance.

d. There is need for Kenyan government to encourage efforts to harvest, treat and reuse wastewater as an alternative source and develop guidelines regarding reuse.

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APPENDICES

Appendix 1: Particle size analysis

Table7.1: Crushed bricks particle size analysis

sieve size(mm)	trial 1	passed	%passed	trial 2	passed	%passed	Average trial	Average passed	%passed
3.35	46	0	0	138	0	0	92	0	0
4.75	29	46	7.849829	61	138	10.1396	45	92	9.450437
5	128	75	12.79863	336	199	14.6216	232	137	14.07293
6.3	383	203	34.64164	826	535	39.30933	604.5	369	37.90447
10	0	586	100	0	1361	100	0	973.5	100

Table 7.2: Charcoal particle size analysis

sieve size(mm)	trial 1	passed	%passed	trial 2	passed	%passed	average trial	average passed	%passed
0	2	0	0	1	0	0	1.5	0	0
0.075	2	2	0.370714	2	1	0.192123	2	1.5	0.283019
0.15	6	4	0.741427	6	3	0.576369	6	3.5	0.660377
0.212	10	10	1.853568	11.5	9	1.729107	10.75	9.5	1.792453
0.3	16	20	3.707136	17	20.5	3.938521	16.5	20.25	3.820755
0.425	18	36	6.672845	15	37.5	7.204611	16.5	36.75	6.933962
0.6	75	54	10.00927	70	52.5	10.08646	72.5	53.25	10.04717
1.18	75	129	23.91103	70	122.5	23.53506	72.5	125.75	23.72642
2	18	204	37.81279	19	192.5	36.98367	18.5	198.25	37.40566
2.83	164	222	41.14921	153	211.5	40.63401	158.5	216.75	40.89623
4.75	16	386	71.54773	17	364.5	70.02882	16.5	375.25	70.80189
5	85	402	74.51344	85	381.5	73.29491	85	391.75	73.91509
6.3	51	487	90.26877	53	466.5	89.62536	52	476.75	89.95283
10	1.5	538	99.72196	1	519.5	99.80788	1.25	528.75	99.76415
12.5	0	539.5	100	0	520.5	100	0	530	100

Appendix 2: Grading curves

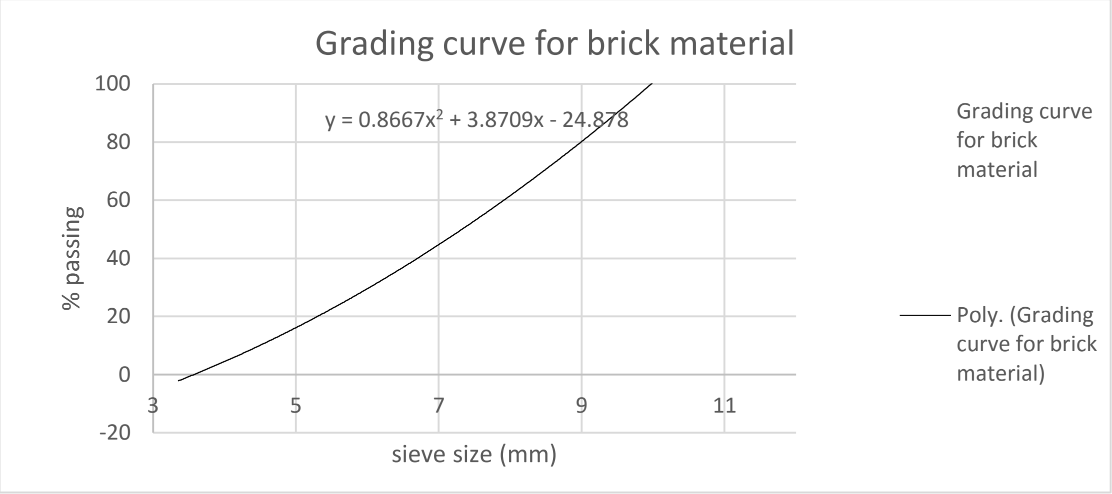

Figure 7.1: Grading curve for crushed brick

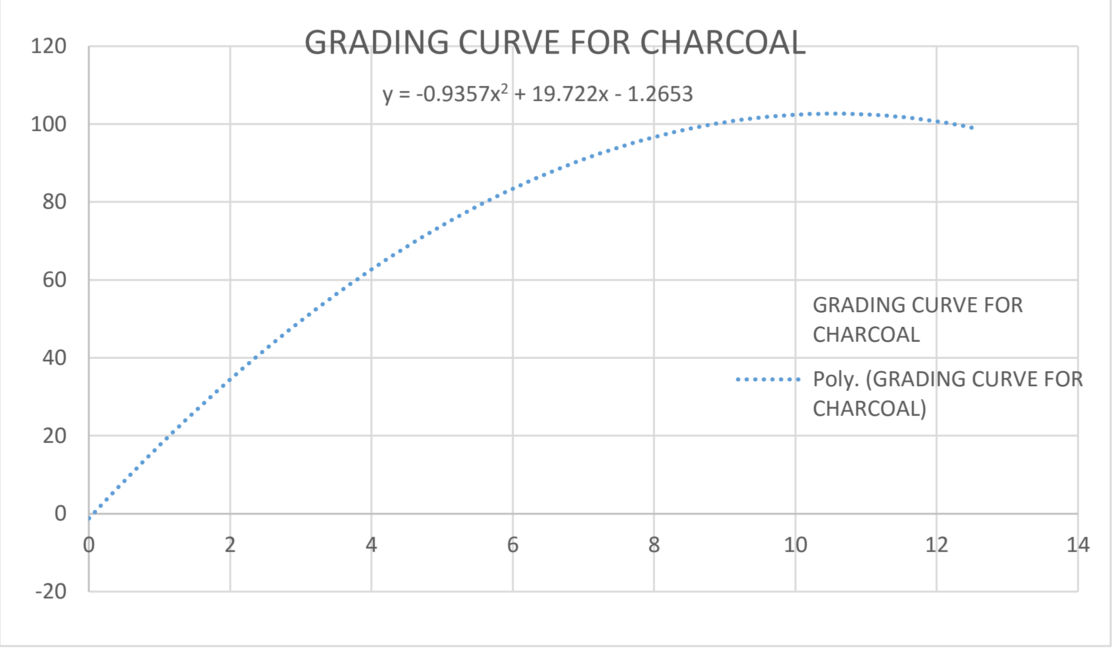

Figure 7.2: Grading curve charcoal

Appendix 3: Standard curves

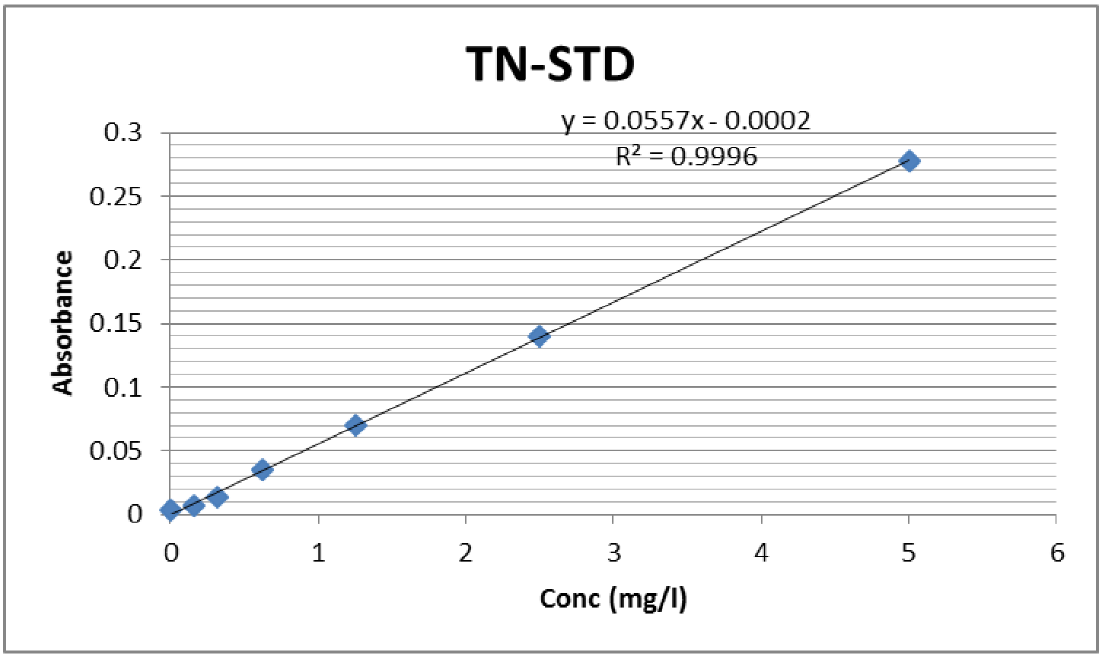

Figure 7.3: Total Nitrogen standard curve

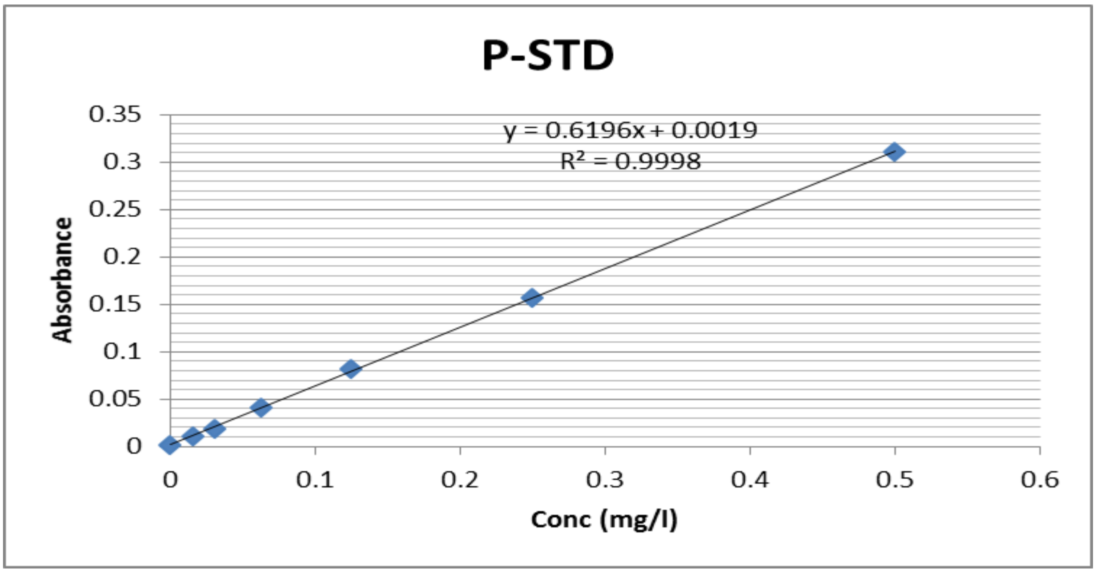

Figure 7.4: Total Phosphorus standard curve

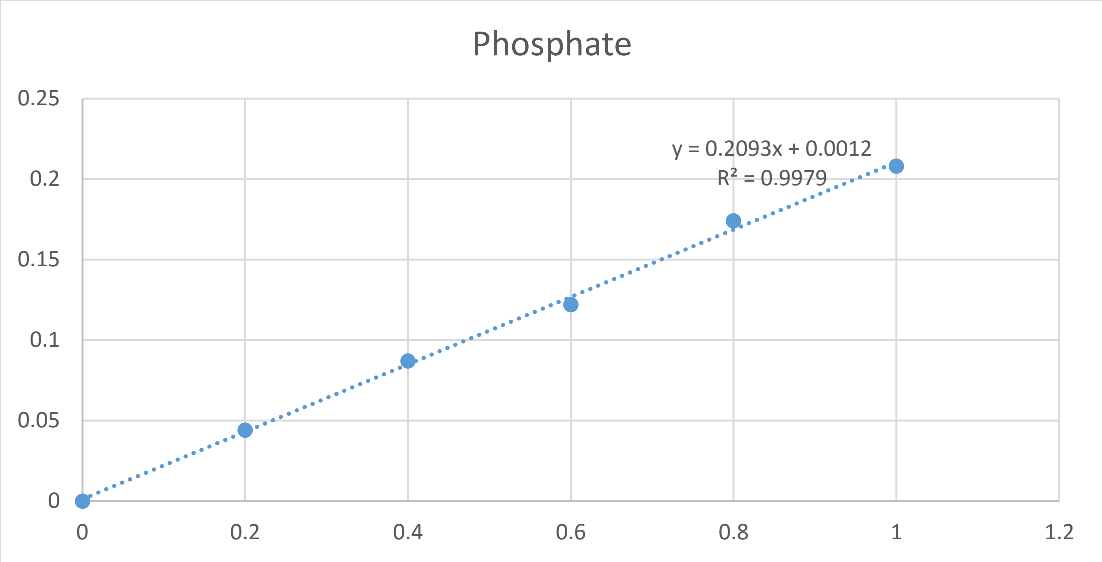

Figure 7.5: Phosphate standard curve

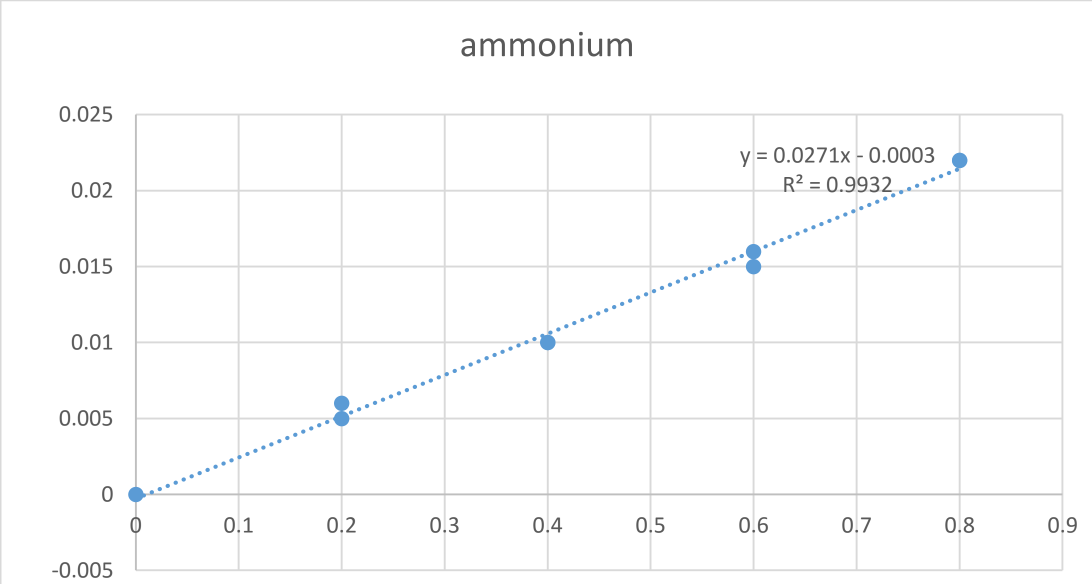

Figure 7.6: Ammonium standard curve.

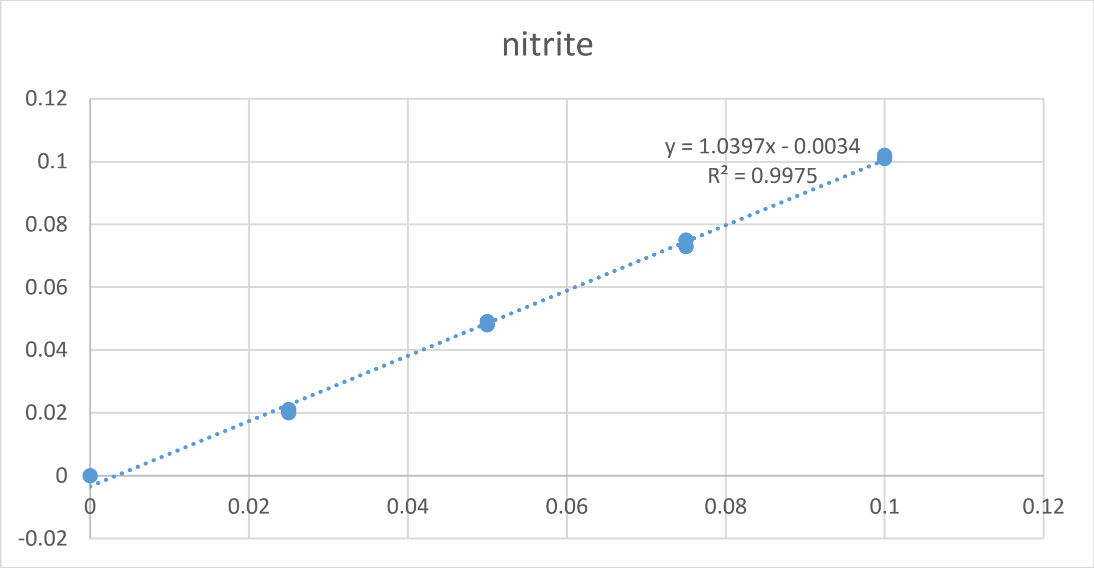

Figure 7.7: Nitrite standard curve

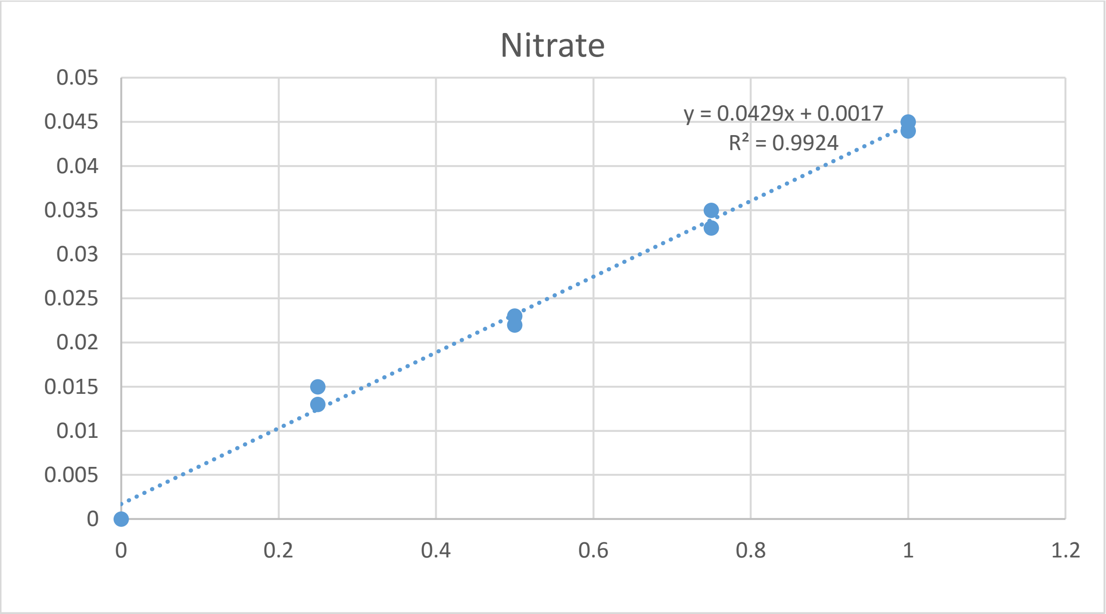

Figure 7.8: Nitrate standard curve

Appendix 4: BOD_5 concentrations

Table 7.3: Concentration trends for BOD5

DATE	WEEK	INFLUENT (RAW W/W)	ANAEROBIC (B + C)	% REDUCTION	ANAEROBIC (B)	% REDUCTION	EFLUENT (B + C)	% REDUCTION FROM ANAEROBIC	TOTAL % REDUCTION	EFLUENT (B)	% REDUCTION FROM ANAEROBIC	TOTAL % REDUCTION
11/6/2015	1	732.21	239.57	67.28124445	232.01	68.31373513	3.12	98.69766665	99.57389274	14.27	93.84940304	98.05110556
18/6/2015	2	386.69	222.85	42.36985699	208.01	46.20755644	2.274	98.97958268	99.41193204	12.276	94.09836066	96.82536399
25/6/2015	3	677.34	242.16	64.24838338	239.66	64.61747424	1.603	99.33804096	99.76333894	11.813	95.07093382	98.25597189
2/7/2015	4	721.62	210.63	70.81150744	204.6	71.6471273	1.738	99.17485638	99.75915302	11.54	94.3597263	98.40082038
9/7/2015	5	473.27	228.6	51.69776238	219.4	53.64168445	1.223	99.46500437	99.74158514	12.26	94.41203282	97.40951254
16/7/2015	6	544.67	177	67.50325885	158.67	70.86859934	1.367	99.22768362	99.74902234	11.82	92.55057667	97.82987864
23/7/2015	7	707.27	162.63	77.00595247	144.3	79.5976077	1.243	99.23568837	99.82425382	14.23	90.13860014	97.98803851
30/7/2015	8	467.42	233.6	50.02353344	236.3	49.44589448	1.653	99.29238014	99.6463566	16.253	93.12187897	96.52282744
6/8/2015	9	622.94	212.6	65.87151251	201.4	67.66943847	1.236	99.41862653	99.80158603	15.236	92.43495531	97.55417857
13/8/2015	10	374.67	227.4	39.3065898	220.7	41.09483012	1.325	99.4173263	99.64635546	16.256	92.63434527	95.66124857
20/8/2015	11	492.63	218.7	55.60562694	212.45	56.87432759	1.743	99.20301783	99.64618476	19.34	90.89668157	96.07413272
27/8/2015	12	555.4	195.2	64.85415916	182.5	67.14079942	2.677	98.62858607	99.51800504	17.26	90.54246575	96.89232985
3/9/2015	13	625	216.5	65.36	209.3	66.512	5.96	97.24711316	99.0464	20.14	90.37744864	96.7776
10/9/2015	14	526.52	224.2	57.41852161	223.4	57.57046266	6.71	97.00713649	98.72559447	16.941	92.41674127	96.78245841
17/9/2015	15	614.2	232.5	62.14588082	226.7	63.09019863	5.85	97.48387097	99.04754152	22.4	90.11910013	96.35297949
24/9/2015	16	378.12	280.03	25.94150005	275.01	27.26912091	4.96	98.2287612	98.68824712	29.88	89.13494055	92.09774675
1/10/2015	17	726.21	294.33	59.47040112	310.3	57.27131271	13.02	95.57639384	98.20713017	39.05	87.41540445	94.62276752
8/10/2015	18	424	268.42	36.69339623	257.84	39.18867925	12.45	95.36174652	97.06367925	20.87	91.90583307	95.07783019
15/10/2015	19	526.21	272.72	48.17278273	269.01	48.87782444	12.94	95.25520681	97.54090572	26.3	90.22341177	95.0019954
22/10/2015	20	482.6	289.36	40.04144219	294.11	39.05719022	14.62	94.94747028	96.97057605	42.3	85.61762606	91.23497721
29/10/2015	21	526.86	237.39	54.94248947	228.41	56.64692708	16.89	92.88512574	96.79421478	39.82	82.56643755	92.44201496
5/11/2015	22	389.42	285.22	26.75774228	247.31	36.49273278	18.92	93.36652409	95.14149248	49.77	79.87545995	87.21945457
AVERAGES		**544.3304545**	**235.0731818**	**54.2510702**	**227.3359091**	**55.86797833**	**6.06918182**	**97.6108095**	**98.78670216**	**21.819318**	**90.62556199**	**95.68523787**

Appendix 5: Absorbance to concentration conversion for the Pollutants

Table 7.4: Conversion of absorbance to concentration for Total Phosphorus

DATE	WEEK	RAW /WW			ANAEROBIC (B+C)			ANAEROBIC (B)			EFFLUENT (B+C)			EFFLUENT (B)		
		ABS.	5 X Dil.	CONC.	ABS.	5 X Dil.	CONC.	ABS.	5 X Dil.	CONC.	ABS.	5 X Dil.	CONC.	ABS.	5 X Dil.	CONC.
11/6/2015	1	1.041	5.205	8.397514526	0.872	4.36	7.03373144	0.88	4.4	7.098289219	0.059	0.295	0.473047127	0.062	0.31	0.497256294
18/6/2015	2	0.9723	4.8615	7.843124597	0.792	3.96	6.388153648	0.802	4.01	6.468850872	0.235	1.175	1.89331827	0.202	1.01	1.627017431
25/6/2015	3	0.982	4.91	7.921400904	0.44	2.2	3.547611362	0.462	2.31	3.725145255	0.095	0.475	0.763557134	0.124	0.62	0.997579083
2/7/2015	4	0.822	4.11	6.63024532	0.641	3.205	5.169625565	0.631	3.155	5.088928341	0.069	0.345	0.553744351	0.109	0.545	0.876533247
9/7/2015	5	0.928	4.64	7.485635894	0.524	2.62	4.225468044	0.502	2.51	4.047934151	0.072	0.36	0.577953518	0.112	0.56	0.900742414
16/7/2015	6	0.738	3.69	5.952388638	0.588	2.94	4.741930278	0.463	2.315	3.733214977	0.081	0.405	0.65058102	0.164	0.82	1.320367979
23/7/2015	7	0.773	3.865	6.234828922	0.433	2.165	3.491123305	0.402	2.01	3.240961911	0.069	0.345	0.553744351	0.102	0.51	0.82004519
30/7/2015	8	0.681	3.405	5.492414461	0.552	2.76	4.451420271	0.456	2.28	3.676726921	0.057	0.285	0.456907682	0.107	0.535	0.860393802
6/8/2015	9	0.805	4.025	6.493060039	0.429	2.145	3.458844416	0.435	2.175	3.50726275	0.066	0.33	0.529535184	0.111	0.555	0.892672692
13/8/2015	10	0.639	3.195	5.15348612	0.32	1.6	2.579244674	0.365	1.825	2.942382182	0.058	0.29	0.464977405	0.109	0.545	0.876533247
20/8/2015	11	0.765	3.825	6.170271143	0.443	2.215	3.571820529	0.404	2.02	3.257101356	0.066	0.33	0.529535184	0.14	0.7	1.126694642
27/8/2015	12	0.507	2.535	4.088282763	0.388	1.94	3.127985797	0.427	2.135	3.442704971	0.056	0.28	0.44883796	0.11	0.55	0.88460297
3/9/2015	13	0.742	3.71	5.984667527	0.412	2.06	3.321659135	0.415	2.075	3.345868302	0.062	0.31	0.497256294	0.132	0.66	1.062136862
10/9/2015	14	0.694	3.47	5.597320852	0.392	1.96	3.160264687	0.401	2.005	3.232892189	0.059	0.295	0.473047127	0.121	0.605	0.973369916
17/9/2015	15	0.99	4.95	7.985958683	0.613	3.065	4.943673338	0.593	2.965	4.78227889	0.072	0.36	0.577953518	0.109	0.545	0.876533247
24/9/2015	16	0.881	4.405	7.106358941	0.621	3.105	5.008231117	0.631	3.155	5.088928341	0.068	0.34	0.545674629	0.129	0.645	1.037927695
1/10/2015	17	0.966	4.83	7.792285345	0.468	2.34	3.773563589	0.593	2.965	4.78227889	0.078	0.39	0.626371853	0.117	0.585	0.941091026
8/10/2015	18	0.607	3.035	4.895255003	0.479	2.395	3.862330536	0.503	2.515	4.056003873	0.078	0.39	0.626371853	0.127	0.635	1.02178825
15/10/2015	19	0.725	3.625	5.847482247	0.582	2.91	4.693511943	0.591	2.955	4.766139445	0.082	0.41	0.658650742	0.135	0.675	1.08634603
22/10/2015	20	0.832	4.16	6.710942544	0.672	3.36	5.419786959	0.669	3.345	5.395577792	0.085	0.425	0.68285991	0.142	0.71	1.142834087
29/10/2015	21	0.721	3.605	5.815203357	0.662	3.31	5.339089735	0.674	3.37	5.435926404	0.08	0.4	0.642511298	0.132	0.66	1.062136862
5/11/2015	22	0.942	4.71	7.598612008	0.597	2.985	4.814557779	0.601	3.005	4.846836669	0.077	0.385	0.61830213	0.135	0.675	1.08634603

Table 7.5: Conversion of absorbance to concentration for Total Nitrogen

DATE	WEEK	RAW /WW			ANAEROBIC (B+C)			ANAEROBIC (B)			EFFLUENT (B+C)			EFFLUENT (B)		
		Abs	Dil. X 10	CONC.	Abs	Dil. X 10	CONC.	Abs	Dil. X 10	CONC.	Abs	Dil. X 10	CONC.	Abs	Dil. X 10	CONC.
11/6/2015	1	0.145185	1.451852	26.06916	0.018148	0.181482	3.261787	0.022685	0.226852	4.076336	0.013611	0.136111	2.447238	0.018148	0.181482	3.261787
18/6/2015	2	0.154259	1.542593	27.69826	0.040833	0.408333	7.334532	0.058981	0.589815	10.59273	0.018148	0.181482	3.261787	0.013611	0.136111	2.447238
25/6/2015	3	0.212333	2.123334	38.12448	0.068056	0.680556	12.22183	0.04537	0.453704	8.149081	0.018148	0.181482	3.261787	0.009074	0.090741	1.632689
2/7/2015	4	0.131574	1.315741	23.62551	0.036296	0.362963	6.519983	0.054444	0.544445	9.778179	0.027222	0.272222	4.890885	0.018148	0.181482	3.261787
9/7/2015	5	0.176944	1.769445	31.771	0.031759	0.317593	5.705434	0.036296	0.362963	6.519983	0.022685	0.226852	4.076336	0.018148	0.181482	3.261787
16/7/2015	6	0.235926	2.35926	42.36014	0.040833	0.408333	7.334532	0.031759	0.317593	5.705434	0.027222	0.272222	4.890885	0.022685	0.226852	4.076336
23/7/2015	7	0.195093	1.950926	35.0292	0.040833	0.408333	7.334532	0.031759	0.317593	5.705434	0.018148	0.181482	3.261787	0.018148	0.181482	3.261787
30/7/2015	8	0.19963	1.996297	35.84375	0.031759	0.317593	5.705434	0.027222	0.272222	4.890885	0.027222	0.272222	4.890885	0.027222	0.272222	4.890885
6/8/2015	9	0.213241	2.132408	38.28739	0.036296	0.362963	6.519983	0.036296	0.362963	6.519983	0.018148	0.181482	3.261787	0.013611	0.136111	2.447238
13/8/2015	10	0.172407	1.724074	30.95645	0.031759	0.317593	5.705434	0.022685	0.226852	4.076336	0.004537	0.04537	0.81814	0.004537	0.04537	0.81814
20/8/2015	11	0.213241	2.132408	38.28739	0.022685	0.226852	4.076336	0.022685	0.226852	4.076336	0.004537	0.04537	0.81814	0.004537	0.04537	0.81814
27/8/2015	12	0.131574	1.315741	23.62551	0.027222	0.272222	4.890885	0.027222	0.272222	4.890885	0.009074	0.090741	1.632689	0.004537	0.04537	0.81814
3/9/2015	13	0.145185	1.451852	26.06916	0.031759	0.317593	5.705434	0.031759	0.317593	5.705434	0.013611	0.136111	2.447238	0.004537	0.04537	0.81814
10/9/2015	14	0.19963	1.996297	35.84375	0.040833	0.408333	7.334532	0.04537	0.453704	8.149081	0.009074	0.090741	1.632689	0.013611	0.136111	2.447238
17/9/2015	15	0.204167	2.041667	36.6583	0.036296	0.362963	6.519983	0.027222	0.272222	4.890885	0.013611	0.136111	2.447238	0.004537	0.04537	0.81814
24/9/2015	16	0.136111	1.361111	24.44006	0.036296	0.362963	6.519983	0.049907	0.499074	8.96363	0.004537	0.04537	0.81814	0.004537	0.04537	0.81814
1/10/2015	17	0.235926	2.35926	42.36014	0.049907	0.499074	8.96363	0.04537	0.453704	8.149081	0.031759	0.317593	5.705434	0.04537	0.453704	8.149081
8/10/2015	18	0.140648	1.406482	25.25461	0.040833	0.408333	7.334532	0.040833	0.408333	7.334532	0.022685	0.226852	4.076336	0.036296	0.362963	6.519983
15/10/2015	19	0.195093	1.950926	35.0292	0.04537	0.453704	8.149081	0.04537	0.453704	8.149081	0.027222	0.272222	4.890885	0.040833	0.408333	7.334532
22/10/2015	20	0.245	2.450001	43.98924	0.063519	0.635185	11.40728	0.058981	0.589815	10.59273	0.036296	0.362963	6.519983	0.031759	0.317593	5.705434
29/10/2015	21	0.131574	1.315741	23.62551	0.054444	0.544445	9.778179	0.054444	0.544445	9.778179	0.040833	0.408333	7.334532	0.040833	0.408333	7.334532
5/11/2015	22	0.186019	1.860186	33.4001	0.04537	0.453704	8.149081	0.04537	0.453704	8.149081	0.031759	0.317593	5.705434	0.036296	0.362963	6.519983

Table 7.6: Conversion of absorbance to concentration for Phosphates

DATE	WEEK	RAW /WW		ANAEROBIC (B+C)		ANAEROBIC (B)		EFFLUENT (B+C)		EFFLUENT (B)	
		ABS.	CONC.	ABS.	CONC.	ABS.	CONC.	ABS.	CONC.	ABS.	CONC.
11/6/2015	1	0.511	2.435738	0.52	2.478739	0.4025	1.917344	0.062	0.290492	0.0285	0.130435
18/6/2015	2	0.4375	2.084568	0.817	3.897754	0.867	4.136646	0.055	0.257047	0.044	0.204491
25/6/2015	3	0.521	2.483516	0.773	3.68753	0.8295	3.957477	0.037	0.171046	0.0345	0.159102
2/7/2015	4	0.506	2.411849	0.9295	4.43526	0.981	4.681319	0.0421	0.195413	0.0395	0.182991
9/7/2015	5	0.4425	2.108457	0.742	3.539417	0.884	4.217869	0.068	0.319159	0.0525	0.245103
16/7/2015	6	0.221	1.050167	0.5995	2.858576	0.6435	3.068801	0.071	0.333493	0.0575	0.268992
23/7/2015	7	0.289	1.37506	0.5305	2.528906	0.566	2.698519	0.0515	0.240325	0.0585	0.27377
30/7/2015	8	0.2455	1.167224	0.511	2.435738	0.565	2.693741	0.059	0.276159	0.055	0.257047
6/8/2015	9	0.36	1.714286	0.6115	2.91591	0.6385	3.044912	0.0515	0.240325	0.15	0.710941
13/8/2015	10	0.2405	1.143335	0.4775	2.275681	0.5375	2.562351	0.039	0.180602	0.1445	0.684663
20/8/2015	11	0.3995	1.90301	0.4275	2.036789	0.427	2.0344	0.063	0.29527	0.09	0.424271
27/8/2015	12	0.333	1.585284	0.503	2.397516	0.5185	2.471572	0.052	0.242714	0.061	0.285714
3/9/2015	13	0.407	1.938844	0.575	2.741519	0.582	2.774964	0.0465	0.216436	0.0525	0.245103
10/9/2015	14	0.382	1.819398	0.608	2.899188	0.591	2.817965	0.044	0.204491	0.053	0.247492
17/9/2015	15	0.629	2.999522	0.702	3.348304	0.698	3.329193	0.0355	0.16388	0.0375	0.173435
24/9/2015	16	0.425	2.024845	0.682	3.252747	0.691	3.295748	0.0475	0.221214	0.057	0.266603
1/10/2015	17	0.641	3.056856	0.766	3.654085	0.8265	3.943144	0.034	0.156713	0.045	0.209269
8/10/2015	18	0.511	2.435738	0.742	3.539417	0.772	3.682752	0.0552	0.258003	0.0571	0.267081
15/10/2015	19	0.489	2.330626	0.685	3.267081	0.675	3.219302	0.0451	0.209747	0.0482	0.224558
22/10/2015	20	0.395	1.88151	0.752	3.587195	0.738	3.520306	0.0501	0.233636	0.0528	0.246536
29/10/2015	21	0.428	2.039178	0.724	3.453416	0.745	3.553751	0.0472	0.21978	0.05	0.233158
5/11/2015	22	0.512	2.440516	0.694	3.310081	0.703	3.353082	0.0411	0.190635	0.0451	0.209747

Table 7.7: Conversion of absorbance to concentration for Ammonium

DATE	WEEK	RAW /WW		ANAEROBIC (B+C)		ANAEROBIC (B)		EFFLUENT (B+C)		EFFLUENT (B)	
		ABS.	CONC.	ABS.	CONC.	ABS.	CONC.	ABS.	CONC.	ABS.	CONC.
11/6/2015	1	0.0355	1.321033	0.0515	1.911439	0.048	1.782288	0.008	0.306273	0.015	0.564576
18/6/2015	2	0.0225	0.841328	0.0355	1.321033	0.046	1.708487	0.013	0.490775	0.012	0.453875
25/6/2015	3	0.0275	1.02583	0.0235	0.878229	0.0305	1.136531	0.003	0.121771	0.0045	0.177122
2/7/2015	4	0.038	1.413284	0.045	1.671587	0.0485	1.800738	0.013	0.490775	0.0135	0.509225
9/7/2015	5	0.0265	0.98893	0.053	1.96679	0.0445	1.653137	0.0145	0.546125	0.0195	0.730627
16/7/2015	6	0.0305	1.136531	0.047	1.745387	0.0465	1.726937	0.016	0.601476	0.0225	0.841328
23/7/2015	7	0.036	1.339483	0.041	1.523985	0.0405	1.505535	0.01	0.380074	0.025	0.933579
30/7/2015	8	0.0455	1.690037	0.048	1.782288	0.05	1.856089	0.013	0.490775	0.019	0.712177
6/8/2015	9	0.026	0.97048	0.039	1.450185	0.0455	1.690037	0.011	0.416974	0.018	0.675277
13/8/2015	10	0.02	0.749077	0.0365	1.357934	0.0365	1.357934	0.011	0.416974	0.0165	0.619926
20/8/2015	11	0.0405	1.505535	0.046	1.708487	0.046	1.708487	0.014	0.527675	0.017	0.638376
27/8/2015	12	0.027	1.00738	0.034	1.265683	0.035	1.302583	0.009	0.343173	0.012	0.453875
3/9/2015	13	0.0305	1.136531	0.042	1.560886	0.0405	1.505535	0.0105	0.398524	0.012	0.453875
10/9/2015	14	0.0365	1.357934	0.048	1.782288	0.04905	1.821033	0.01	0.380074	0.013	0.490775
17/9/2015	15	0.0495	1.837638	0.0405	1.505535	0.0495	1.837638	0.0135	0.509225	0.0135	0.509225
24/9/2015	16	0.0374	1.391144	0.042	1.560886	0.042	1.560886	0.016	0.601476	0.013	0.490775
1/10/2015	17	0.042	1.560886	0.052	1.929889	0.055	2.04059	0.016	0.601476	0.0155	0.583026
8/10/2015	18	0.035	1.302583	0.049	1.819188	0.0485	1.800738	0.016	0.601476	0.015	0.564576
15/10/2015	19	0.028	1.04428	0.0395	1.468635	0.038	1.413284	0.017	0.638376	0.018	0.675277
22/10/2015	20	0.0375	1.394834	0.046	1.708487	0.049	1.819188	0.016	0.601476	0.017	0.638376
29/10/2015	21	0.0425	1.579336	0.0505	1.874539	0.049	1.819188	0.018	0.675277	0.0185	0.693727
5/11/2015	22	0.0285	1.062731	0.0385	1.431734	0.0405	1.505535	0.0165	0.619926	0.017	0.638376

Table 7.8: Conversion of absorbance to concentration for Nitrites

DATE	WEEK	RAW /WW		ANAEROBIC (B+C)		ANAEROBIC (B)		EFFLUENT (B+C)		EFFLUENT (B)	
		ABS.	CONC.	ABS.	CONC.	ABS.	CONC.	ABS.	CONC.	ABS.	CONC.
11/6/2015	1	0.049	0.050399	0.027	0.029239	0.037	0.038857	0.0546	0.055785	0.342	0.332211
18/6/2015	2	0.041	0.042705	0.021	0.023468	0.0205	0.022987	0.0515	0.052804	0.295	0.287006
25/6/2015	3	0.043	0.044628	0.025	0.027316	0.023	0.025392	0.0525	0.053766	0.237	0.231221
2/7/2015	4	0.0405	0.042224	0.018	0.020583	0.0155	0.018178	0.0485	0.049918	0.355	0.344715
9/7/2015	5	0.0275	0.02972	0.024	0.026354	0.025	0.027316	0.0775	0.077811	0.358	0.3476
16/7/2015	6	0.0345	0.036453	0.0225	0.024911	0.027	0.029239	0.0796	0.079831	0.6385	0.61739
23/7/2015	7	0.0335	0.035491	0.0355	0.037415	0.0305	0.032606	0.0695	0.070116	0.6015	0.581802
30/7/2015	8	0.0562	0.057324	0.041	0.042705	0.0425	0.044147	0.055	0.05617	0.649	0.627489
6/8/2015	9	0.0195	0.022026	0.0225	0.024911	0.0245	0.026835	0.0935	0.0932	1.1575	1.116572
13/8/2015	10	0.0275	0.02972	0.02	0.022506	0.0145	0.017217	0.0574	0.058478	0.2765	0.269212
20/8/2015	11	0.0465	0.047995	0.0303	0.032413	0.03	0.032125	0.0595	0.060498	0.525	0.508224
27/8/2015	12	0.0345	0.036453	0.023	0.025392	0.023	0.025392	0.0785	0.078773	0.2535	0.247091
3/9/2015	13	0.0425	0.044147	0.0294	0.031548	0.0287	0.030874	0.0628	0.063672	0.194	0.189862
10/9/2015	14	0.0382	0.040012	0.0341	0.036068	0.0336	0.035587	0.0942	0.093873	0.728	0.703472
17/9/2015	15	0.0355	0.037415	0.0315	0.033567	0.027	0.029239	0.0975	0.097047	1.03	0.993941
24/9/2015	16	0.0335	0.035491	0.0275	0.02972	0.0335	0.035491	0.0852	0.085217	0.794	0.766952
1/10/2015	17	0.0265	0.028758	0.019	0.021545	0.0215	0.023949	0.0925	0.092238	0.1675	0.164374
8/10/2015	18	0.0315	0.033567	0.0232	0.025584	0.0225	0.024911	0.0622	0.063095	0.1245	0.123016
15/10/2015	19	0.0385	0.0403	0.0208	0.023276	0.0221	0.024526	0.0742	0.074637	0.184	0.180244
22/10/2015	20	0.0291	0.031259	0.0331	0.035106	0.0328	0.034818	0.0682	0.068866	0.0882	0.088102
29/10/2015	21	0.0341	0.036068	0.0301	0.032221	0.0314	0.033471	0.0729	0.073387	0.129	0.127344
5/11/2015	22	0.0291	0.031259	0.0302	0.032317	0.0295	0.031644	0.0819	0.082043	0.1029	0.102241

Table 7.9: Conversion of absorbance to concentration for Nitrates

DATE	WEEK	RAW /WW		ANAEROBIC (B+C)		ANAEROBIC (B)		EFFLUENT (B+C)		EFFLUENT (B)	
		ABS.	CONC.	ABS.	CONC.	ABS.	CONC.	ABS.	CONC.	ABS.	CONC.
11/6/2015	1	0.052	1.172494	0.0445	0.997669	0.0475	1.067599	0.2205	5.100233	1.9685	45.84615
18/6/2015	2	0.0475	1.067599	0.034	0.752914	0.0305	0.671329	0.194	4.482517	0.675	15.69464
25/6/2015	3	0.025	0.543124	0.015	0.310023	0.005	0.076923	0.1865	4.307692	0.436	10.12354
2/7/2015	4	0.032	0.706294	0.0195	0.414918	0.0175	0.368298	0.1945	4.494172	0.469	10.89277
9/7/2015	5	0.0265	0.578089	0.0245	0.531469	0.027	0.589744	0.288	6.67366	0.6105	14.19114
16/7/2015	6	0.0255	0.554779	0.0275	0.601399	0.0315	0.694639	0.3125	7.244755	0.6795	15.79953
23/7/2015	7	0.043	0.962704	0.027	0.589744	0.0242	0.524476	0.2145	4.960373	1.8655	43.44522
30/7/2015	8	0.058	1.312354	0.039	0.869464	0.0435	0.974359	0.2225	5.146853	1.059	24.64569
6/8/2015	9	0.03	0.659674	0.016	0.333333	0.018	0.379953	0.2855	6.615385	1.469	34.2028
13/8/2015	10	0.0295	0.648019	0.0205	0.438228	0.0185	0.391608	0.27	6.254079	1.3795	32.11655
20/8/2015	11	0.0415	0.927739	0.026	0.566434	0.0245	0.531469	0.1895	4.377622	0.8625	20.06527
27/8/2015	12	0.035	0.776224	0.0335	0.741259	0.0365	0.811189	0.2565	5.939394	0.758	17.62937
3/9/2015	13	0.0371	0.825175	0.021	0.449883	0.0201	0.428904	0.241	5.578089	1.291	30.05361
10/9/2015	14	0.0394	0.878788	0.0295	0.648019	0.0215	0.461538	0.272	6.300699	2.193	51.07925
17/9/2015	15	0.0395	0.881119	0.034	0.752914	0.0325	0.717949	0.2515	5.822844	2.254	52.50117
24/9/2015	16	0.0295	0.648019	0.03	0.659674	0.032	0.706294	0.264	6.114219	0.9595	22.32634
1/10/2015	17	0.028	0.613054	0.027	0.589744	0.0305	0.671329	0.295	6.83683	0.725	16.86014
8/10/2015	18	0.0314	0.692308	0.029	0.636364	0.02705	0.590909	0.231	5.344988	0.995	23.15385
15/10/2015	19	0.0292	0.641026	0.0315	0.694639	0.0305	0.671329	0.279	6.463869	1.042	24.24942
22/10/2015	20	0.0317	0.699301	0.03	0.659674	0.0305	0.671329	0.225	5.205128	1.387	32.29138
29/10/2015	21	0.0305	0.671329	0.031	0.682984	0.03	0.659674	0.324	7.512821	1.629	37.9324
5/11/2015	22	0.0355	0.787879	0.0295	0.648019	0.0255	0.554779	0.294	6.81352	1.335	31.07925

Appendix 6: Plagiarism Awareness Certificate

SR701

ISO 9001:2019 Certified Institution

THESIS WRITING COURSE

PLAGIARISM AWARENESS CERTIFICATE

This certificate is awarded to

GEORGE OTIENO ONYANGO

TEC/PGCS/04/12

In recognition for passing the University's plagiarism

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