Performance assessment of a wastewater treatment plant in Kumasi, Ghana

Eivind Bigum Kvernberg

Norwegian University of Life Sciences
Department of Mathematical Sciences and Technology
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Aknowlegements

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Eivind Bigum Kvernberg

Abstract

Developing countries experience a huge gap in the coverage of collection and treatment of domestic wastewater. Where wastewater treatment facilities exist, they often work below design standards. This leads to the discharge of pollutants into natural water bodies, creating a negative impact on the environment and human health.

In this study, a performance assessment of the Kwame Nkrumah University of Science and Technology (KNUST) wastewater treatment plant in Kumasi, Ghana was carried out from February to March 2012. The main objective of the study was to assess the performance of the treatment plant with respect to the removal of microbial and chemical pollutants. Daily samples were collected at critical treatment steps of the plant, and analysed for chemical, physical and microbial parameters.

The study showed that the KNUST wastewater treatment plant is running sub-optimally in the removal of pollutants harmful to the environment and human health. The concentrations of E. coli, TFC and BOD all exceeded the benchmark concentration levels acceptable to the Environmental Protection Agency (EPA) of Ghana. The poor removal of E. coli/TFC (0.55 log) was particularly alarming, as it proved that the plant does little to reduce microbial health risks. Overloading of the plant beyond its design capacity and poor maintenance practices were identified to be the main causes of the plant's poor performance.

Given the cost of running the plant, it is essential that improvements are made to increase the performance. Possible improvements must as a minimum follow criteria such as low investment and maintenance costs, increase of the plant's hydraulic capacity and be easy to operate and maintain.

Sammendrag

Mangelfull oppsamling og behandling av avløpsvann er et stort problem i de fleste utviklingsland. Der eksisterende rensefasiliteter finnes, oppfyller de ofte ikke designkriteriene. Dette medfører store utslipp av stoffer i naturlige vannforekomster som gir en negativ påvirkning på både miljø og sanitærforhold.

I dette studiet ble en ytelsesvurdering av 'Kwame Nkrumah University of Science and Technology (KNUST) wastewater treatment plant' i Kumasi, Ghana gjennomført fra februar til mars 2012. Hovedmålet var å vurdere anleggets renseeffekt på mikrobielle og kjemiske forurensningsstoffer. Daglige prøver ble samlet inn på kritiske punkter i anlegget, før disse ble analysert med hensyn på avløpsvannets kjemiske, fysiske og mikrobielle innhold.

Studiet viste at anlegget opererer med en utilfredsstillende grad av renseeffekt, som medfører at utslippsvannet utgjør en fare for både miljøet og menneskelig helse. Konsentrasjonene av både E. coli, TFC og BOD overskred veiledende akseptable utslippsverdier gitt av Environmental Protection Agency (EPA) i Ghana. Den lave rensegraden av E. coli og TFC (0.55 log reduksjon) var spesielt alarmerende, da det beviser at anlegget ikke utgjør et stort bidrag for å redusere smittefare. Hovedgrunnene til den svake ytelsen på anlegget kan knyttes til en kombinasjon av kraftig hydraulisk overbelastning og dårlige vedlikeholdsrutiner.

Gitt kostnadene ved å drive anlegget, er det avgjørende at forbedringer blir iverksatt for å øke ytelsen. Mulige forbedringer må som et minimum oppfylle kriterier som lave investerings- og vedlikeholdskostnader, økning av anleggets hydrauliske kapasitet og være lett å drifte og vedlikeholde.

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Abbreviations

ANN	Artificial Neural network
BOD	Biological oxygen demand
DO	Dissolved oxygen
E. coli	Escherichia coli
EPA	Environmental Protection Agency
KNUST	Kwame Nkrumah University of Science and Technology
MPN	Most probable number
TFC	Total fecal coliforms
TSS	Total suspended solids
UNICEF	The United Nations Children’s Fund
WHO	World Health Organization
WWTP	Wastewater treatment plant

1.0 Introduction

1.1 Background

Poor sanitation condition is widespread across many developing countries. In 2012, only 30 % of the population in Sub-Saharan Africa had access to improved sanitation (WHO/UNICEF 2012). The main improved sanitation systems were dominated by on-site installations designed for the collection and storage of human excreta such as pit latrines. Systems that ensured the collection, transportation and proper treatment of wastewater remained very low. For instance, across the cities of most developing countries, less than 15% of the collected wastewater is treated before discharge (Mara 2003). The situation is particularly worse in Sub-Saharan Africa. According to United Nations Development Programme (UNDP), in 2000 only 2 % of the cities in sub-Saharan Africa had wastewater treatment facilities and only 30 % of these were operating satisfactorily.

The cities of Ghana are no exception to the poor wastewater treatment coverage. It has been shown that out of the 44 wastewater treatment plants in Ghana, only 20 % are working, most of them below design standard (IMWI 2012). Thus raw wastewater is discharged into the urban sphere with severe consequences on the environment and human health. Generally, the poor coverage of wastewater treatment facilities is attributed to a number of factors, including but not limited to lack of funds, ignorance of low-cost wastewater treatment processes and economic benefits of treated wastewater re-use, together with the tendency among decision-makers to accept the status quo: the continued discharge of untreated wastewater into the environment (Mara 2003). Understanding these inhibitory factors is critical for the planning, design and implementation of an effective urban wastewater management system. However, studies on the exploration of these factors in relation to wastewater treatment plants in developing countries are limited. This limits the range of options for optimizing the performance of existing wastewater treatment plants.

1.2 Aim and objectives

The main aim of this study was to assess the overall performance of a wastewater treatment plant, and identify factors inhibiting its performance. In line with the main objective, the specific objectives were to:

- Assess the performance of the treatment steps of the wastewater treatment plant in relation to the removal of microbial and chemical quality parameters;
- Identify performance limiting factors in the treatment steps of the wastewater treatment plant;
- Recommend strategies for optimizing the performance of the wastewater treatment plant.

2.0 Literature review

2.1 Historical background on wastewater treatment

The existence of wastewater and the need for wastewater treatment is not a new problem. The production of excreta and urine is a natural part of human life, and has a history as long as mankind. In parallel to growing civilizations and increasing urbanity, and with the introduction of the water closet and centralized wastewater collection, problems related to large accumulations of wastewater has arisen. In centralized systems for wastewater collection one could also find other sources to wastewater than only domestic, such as storm water and industrial wastewater sources.

Wastewater is generally looked upon as a negative resource, both from an aesthetic perspective and because of its characteristic bad odor, and the fact that its main component is human waste. Of greater importance when considering the need for treatment is the fact that untreated wastewater led into a natural water body constitute a great hazard for the environment and a health risk for human and animal life. The environmental risk is mainly due to overloading of physical and chemical components associated with human activity into an aquifer, while the health risk is mainly the result of pathogenic contamination.

The problem of the contamination of water bodies through wastewater discharges was understood back in the time of the Romans. The first sewer in Rome was built about 400 BC under the name Cloaca Maxima ('Great Sewer'), a system mainly for transportation of drainage water. During the middle ages there was little progress in urban drainage and sewerage, until the introduction of water closets in the early 19th century. At first these were usually connected to cesspools instead of sewers. In parallel with growing population density in urban areas, and problems of overflowing of cesspools, the problem of wastewater discharge became intolerable. Another factor that attracted the attention to the need of wastewater collection and treatment was the global cholera outbreaks in the 19th century. The disease was gradually traced back to well-water supplies contaminated with human waste from cesspools and privy vaults. As a result of this development, water closets in larger towns were to a larger extent connected to storm sewers. On the other hand, the handling of one problem led to the introduction of another one: surface water pollution.
A receiving water body will up to a certain level be able to render harmless the contaminants of discharged wastewater through dilution. Nevertheless, when the quantities of pollutants exceed the recipients critical level, they will possibly do harm to the surroundings. In densely populated areas this is much likely to happen. The solution to this problem is through treatment of the raw wastewater.

During the late 19th and the early 20th century, there was an awakening in the development of centralized wastewater treatment systems, mainly in the United Kingdom and the United

States. As an addition to collection and discharge of wastewater, physical, biological and chemical processes for the wastewater treatment were introduced, for the removal of pollutants. The idea of separated systems also sprung up at this time, as mixing of storm water and domestic wastewater lead to overloading of the treatment plants. Through the 20th century, there was an increasing public concern for environmental issues, leading to a wider focus on wastewater disposal practices (*Britannica, 2012*). More advanced treatment techniques were developed, tailored for specific constituents in the wastewater. At conventional treatment plants, tertiary treatment steps for removal of nutrients contributing to eutrophication have been widely introduced where the recipient is especially vulnerable. Treatment processes designed for different types of industrial wastewater has also been developed to a large extent. Today, most geographical areas have national regulations for maximum discharge values of different constituents, determining the scope of treatment necessary

2.2 Wastewater treatment in developing countries

According to the UNICEF/WHO, only 30 % of the population in Sub-Saharan Africa has access to improved sanitation (UNICEF/WHO 2012). Trends from 1990 to 2010 shows that increases in access to improved sanitation has been lowest in Sub-Saharan Africa at 4 % (UNICEF/WHO 2012). Figure 1 gives a visual presentation of sanitation coverage in the countries of the world, and highlights the fact that the southern part of the world suffers from low sanitation coverage.

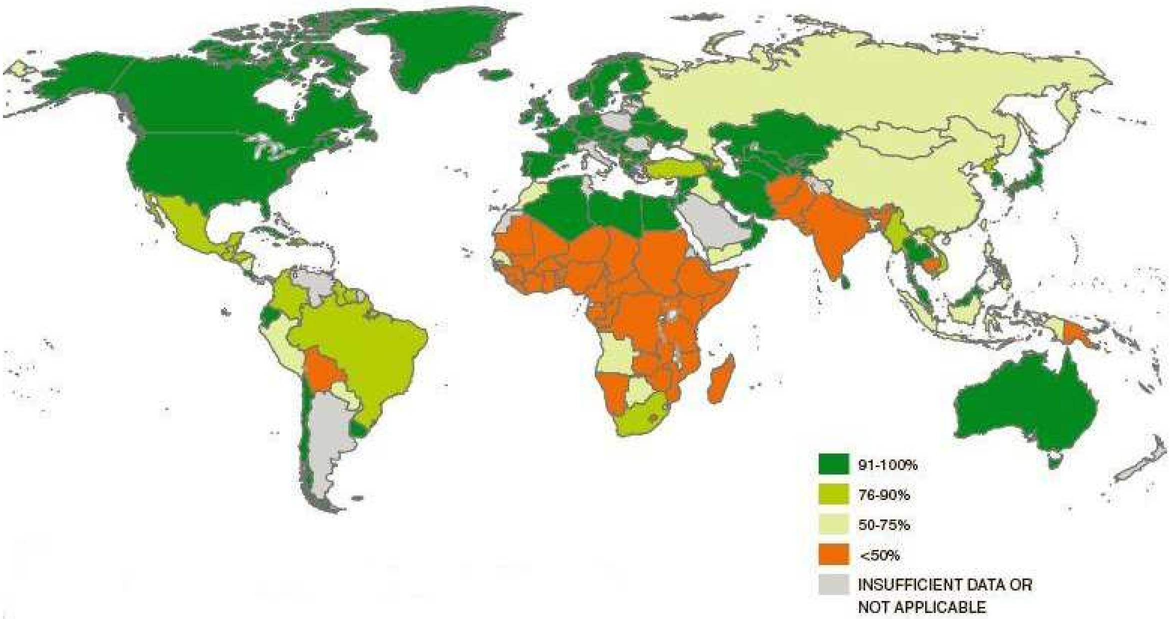

Figure 1: Proportion of the population using improved sanitation in 2010 (UNICEF/WHO 2012)

The MDG focuses extensively on subjects such as infectious disease prevention, hygiene, and providing health and livelihood improvements (Tsuzuki 2012). Directly related to this is the need to reducing excreted pathogens discharge into the environment through wastewater treatment coverage. In most developing countries the coverage of wastewater treatment remains low. In most developing countries, less than 15% of the collected wastewater is treated before discharge (Mara 2003). According to United Nations Development Programme (UNDP), in 2000 only 2 % of the cities in sub-Saharan Africa had sewage treatment and only 30 % of these were operating satisfactorily. In Asia, countries such as Iran, Yemen and Oman only had 10-30 % of the population connected to public collection systems. Septic tanks and cesspools are alternative systems in these areas, while latrine pits or small-bore sewers are widely used in rural areas (UNEP 2000). Mara (2003) describes a complex set of reasons for the insufficient level of wastewater treatment coverage in the developing countries. These reasons include lack of funds, ignorance of low-cost wastewater treatment processes and economic benefits of treated wastewater re-use, together with the tendency among decision-makers to accept the status quo: the continued discharge of untreated wastewater into the environment.

2.2.1 Wastewater treatment in Ghana

According to the WHO/UNICEF (2000) only 14 % of Ghana's population have access to improved sanitation facilities while an additional 58 % use shared sanitation facilities (explained as 'sanitation facilities of an otherwise acceptable type that is shared between two or more households, including public toilets) (Unicef/WHO 2012). Shared sanitation facilities are not considered as improved out of concerns of cleanliness and accessibility. The rest of the population is shared between those having 'unimproved facilities' (9 %) and 'open defecation' (19 %).

In general, waste water disposal is a major challenge in urban areas of Ghana. In the report 'Feasibility Study- Evaluation of the Faecal Sludge and Waste Water Treatment Plants' written by the International Water Management Institute (IWMI) in 2008, it is stated that Ghana has a total of 44 wastewater treatment plants, with only 20 % of these working, most of them below design standard (IWMI 2008). The most widely used treatment options in the country include waste stabilization ponds, trickling filters and activated sludge processes. Since many of these treatment plants are broken down or working sub-optimally, large quantities of wastewater is discharged directly into the recipient, causing a negative impact on the environment.

Some studies have made on the performance of wastewater treatment plants in recent years. W. Kagya wrote a master's thesis in 2009, comparing effluent quality of two wastewater treatment systems in Juapong; one activated sludge treatment plant and one system of stabilization ponds. The main findings were that both systems achieved satisfactory treatment efficiency on most parameters, but failed to meet the EPA benchmark values for E. coli and nutrients such as nitrate and nitrite (Kagya 2009). A similar study was done on an upflow anaerobic sludge blanket (UASB) sewage treatment plant at James Town (Accra) in 2008. The study showed good results for BOD removal, but the effluent did exceed the EPA effluent limits for ammonia and nitrate (Awuah & Abrokwa 2008). Microbial parameters were not investigated in this study. Also, a study on the KNUST wastewater treatment plant was done by A. Fosu in 2009, assessing the efficiency of the plant. Effluent values were found to be higher than the EPA permitted values for TSS, BOD, nitrate, E. coli and TFC. A combination of operational, administrative and design factors was listed as reasons for the poor performance.

In the report from IWMI, several possible reasons for the problems of the Ghanaian wastewater treatment plants are suggested. These can be divided into three main groups; technical issues, institutional issues and financial issues. The technical issues include damage and wear and tear on physical components of the plants, blocked sewer lines, power cuts and more. The institutional issues are related to inadequate operation and maintenance activities, lack of qualified personnel, lack of commitment of the authorities in charge and a general lack of motivation among workers. The financial issues deal with lack of funds to buy items for maintenance and repair works and poor remuneration of workers at the plants.

2.3 Characteristics of domestic wastewater

In a given geographic area or community, several types of wastewater are possibly led to the WWTP through the collection system. The components found in a wastewater flow depend on the type of collection system that is used. The major flow of wastewater to a conventional WWTP is normally *domestic wastewater*, which refers to 'wastewater discharged from residences and from commercial, institutional, and similar facilities'(Metcalf & Eddy 2004). This includes both blackwater (mainly fecal matter and urine) and greywater (mainly water from domestic dishwashing, laundry and bathing). *Industrial wastewater* could also be led into the collection system, even though many industries have their own treatment facilities for their wastewater. Where large scale industries have their wastewater connected to the collection system, it is of great importance to know the characteristics of the wastewater, as it may contain chemical or physical components that might interrupt conventional processes. *Infiltration water* refer to water that enters the collection system through indirect means, principally through the ground and via leakages in the collection system. Cracks, porous walls or joints in the piping system together with other weak points in the system, such as manhole walls are a main reason for this. *Inflow* is the water that are discharged directly into the system through service connections such as cellar and foundation drains, cooling water discharges, and drains from springs and swampy areas. The last category, *storm water*, comes directly into the collection system as runoff from rainfall or possibly snowmelt.

Collection systems are normally divided into three main categories. Separate systems are divided into two types. *Sanitary systems* consist of domestic wastewater and possibly industrial wastewater and infiltration/inflow. Separate systems that only consist of storm water collection are named *Storm collecting systems.* There are also *combined* systems, where all the different types of wastewater are collected and led to a WWTP (Metcalf & Eddy 2004).

The constituents of the domestic wastewater can be divided into physical, chemical and biological parameters which are in many ways interrelated and are all important in the matter of treatment performance, environmental impact, reuse potential and health aspects. The most significant constituents and properties of wastewater are described in chapter 2.4 and 2.5.

2.4 Physical/chemical parameters

Temperature

Generally, wastewater has a higher temperature than the local water supply, as a result of a high content of warm water from households or industries. The mean temperature values varies with the local air temperatures. In the United States the mean annual temperatures of wastewater varies from about 3 to 27°C, while temperatures for some countries in Africa and the Middle East has been reported as high as 30 to 35°C. The optimum temperature for biological treatment is in the range of 25 and 35 °C. For lower temperatures the microbial reactions will appear more slowly, and at very high temperatures, aerobic digestion and nitrification stop. Effluent water with higher temperatures than naturally found in the resipient could also affect the conditions regarding aquatic life, as it can cause a change in the species of fish that can live there (Metcalf & Eddy 2004).

Hydrogen-ion concentration (pH)

pH refer to the negative logarithm of the hydrogen-ion concentration, expressed as:

$$pH = - \log_{10} [H^+]$$

Since most microbial life occur within a narrow pH range (typically 6-9), the hydrogen-ion concentration is of great concern in relation to biological treatment. Influent water with exceptional high or low pH-values (typically industrial wastewater) can be hard to treat by biological means. Effluent water may also affect the pH of the natural waters in the recipient.

Dissolved oxygen (DO)

Dissolved oxygen is essensial for all kinds of aerobic life forms. In aerobic biological wastewater treatment, dissolved oxygen in the water is required for bacterial respiration. Environmental conditions, such as temperature, partial pressure of the gas in the atmosphere, the solubility of the gas and the concentration of impurities such as salts, suspended solids etc. all affect the quantity of oxygen that can be present in a solution. The comparison of DO-concentration before and after biological treatment steps are thus of great interest, since it indicates the rate of biological activity within the treatment unit.

Total suspended solids (TSS)

TSS gives an indication of the content of solid matter in the wastewater. In general, raw wastewater contains solids of variable types and sizes. Larger objects and course materials are normally removed in the first stage of the treatment process. TSS derives from the total solids content (TS) which cover all types of solids found in a wastewater flow, normally a mixture of floating matter, settleable matter, collodial matter and matter in solution. Typicaly, 60 % of the suspended solids are settleable. TSS values are widely used to determine treatment efficiency for conventional treatment processes and to assess the need for effluent filtration in the case of reuse applications (Metcalf & Eddy 2004).

Organic content in wastewater

The level of organic compounts are widely used as a measure of contamination in wastewater, and to evaluate the performance of conventional treatment processes. The organic content is usually measured as *biochemical oxygen demand*, *chemical oxygen demand* or *total organic carbon*.

Biochemical Oxygen Demand (BOD): BOD is a measure of the concentration of biodegradable substances in the wastewater, normally composed of a combination of carbon, hydrogen, oxygen and nitrogen. These substances are broken down by energy-consuming bacteria, and can be measured by detecting the amount of oxygen that are used over a period of 5 or 7 days.

Chemical Oxygen Demand (COD): COD is a measure of the concentration of the contaminants in the water that can be oxidised by a chemical oxidising agent (Kemira 2003). Dichromate in acid solution is used as the oxidising agent. Even though one should expect the values of BOD and COD to be the same, COD values are normally higher. Some of the reasons for this is as follows: (1) Some organic substances can be oxidised chemically, but are harder to oxidize biologically. Lingin is an example of this. (2) The oxidising agent, dichromate, oxidises some inorganic substances that increases the value of apparent organic content. (3) Some organic substances could possibly be toxic to the microorganisms used in the BOD test.

Total Organic Carbon (TOC): TOC is also a measure of the content of organic matter, and is determined by burning a sample and measuring the amount of carbon dioxide which is generated.

Total nitrogen

Raw domestic wastewater normally holds a large fraction of nitrogen, either as organically bonded nitrogen or in inorganic forms such as ammonium (NH_4^+), nitrite (NO_2^-) or nitrate (NO_3^-). The term *total nitrogen* refer to the sum of the organic and inorganic compounds of nitrogen. When the term *Kjeldahl nitrogen* is used, it refer to the sum of organic nitrogen and inorganic nitrogen from ammonium. Urea and proteins are normally the main contributors to the nitrogen content in raw wastewater. Nitrogen is an essential nutrient for the growth of microorganisms, plants and animals. Since it is an essensial building block in the synthesis of protein, it is a necessity in biological treatment processes. The content of nitrogen in the effluent of wastewater cause an environmental concern, as it contributes to eutrophication. On the other hand, if reuse of the wastewater effluent for irrigation is desirable, the nitrogen content should be conserved as it makes an important nutrient for this purpose.

Ammonia

Decomposition by bacteria changes the organic form of nitrogen to ammonia, and the relative amount of ammonia present in the wastewater is thus an indicator of the age of the wastewater.

Nitrate

In aerobic environments, bacteria oxidize the ammonia nitrogen to nitrites and nitrates. The predominance of nitrate nitrogen in wastewater indicates that the wastewater has been stabilized with respect to oxygen demand.

Phosphorus

Just like nitrogen, phosphorus is an essential nutrient for growth of biologocal life. Raw wastewater normally holds a large fraction of phosphorus, and as it makes a significant contribution to eutrophication when led untreated into a natural water body, it should be removed during treatment. Phosphorus is, just like nitrogen, of great interest in relation to reuse purposes, since it constitute a resource that can be utilized for irrigation means.

2.5 Biological parameters

The microorganisms present in wastewater treatment plants are of great importance considering the degradation of organic matter in biological treatment. On the other hand, pathogenic microbial agents, which can cause diseases to humans, are present in large numbers in untreated wastewater, mainly through the content of human excreta. An important objective of wastewater treatment is to reduce the level of pathogenic microorganisms, and thereby reduce the health risk related to discharge of the effluent and reuse of biosolids from wastewater treatment.

In developing countries, analysis of pathogenic organisms remains a challenge due to limited laboratory capacity. Thus, indicator organisms such as total faecal coliforms and E.coli have been widely used to assess the performance of wastewater treatment plants. Fecal coliforms is subgroup of coliforms, and are often used as an indicator of fecal contamination in water. In comparison to coliforms, the fecal coliforms exclude those coliforms that are not solely enteric bacteria and are more commonly found in plant and soil samples.
E. coli is the predominant form of fecal coliforms, and are found in the faeces of warmblooded animals. They are historically used as an indicator of fecal content in environmental samples. E. coli can cause gastroenteritis, which has the symptom of diarrhea in humans.

2.6 Critical effluent parameters

Wastewater contains physical, chemical and microbial parameters that can negatively impact on the environment and human health if discharged to a natural water body without treatment. These constituents can be divided into those causing an environmental hazard and those causing a hazard for human health.

The environmental hazards are to a large extent related to eutrophication. Eutrophication occurs when a natural water body is overloaded with phosphorus and nitrogen, causing extensive algal growth. Decomposition of algae requires large amounts of oxygen. This gives less available oxygen in the water body, causing fish death (Kemira 2003).

Health hazards are associated with pathogenic microbial agents from wastewater that are not removed before the wastewater is discharged into the environment. The greater the quantity of pathogenic agents transmitted to the environment, the greater are the risk of disease outbreaks.

The maximum permitted discharge values of critical parameters in the wastewater are normally given by national regulations. The Environmental Protection Agency (EPA) of Ghana has given the guidelines for Ghana as shown in table 1.

Table 1: EPA Ghana standards for maximum permissible wastewater effluent discharge levels (EPA 2000)

Parameter	EPA Guideline Value
pH	6-9
Temperature (° C)	< 3 °C above ambient
TSS (mg/l)	50
BOD_5 (mg/l)	50
Ammonia (mg/l)	1.0
Nitrate (mg/l)	50
Total Phosphorus (mg/l)	2.0
E. coli (MPN/100 ml)	0
Total Coliforms (MPN/100 ml)	400

WHO use less stringent guidelines on E. coli discharge levels, as a maximum of 1000 MPN/100 ml is accepted (WHO 2006).

2.7 Wastewater treatment processes

The following section reviews the functions of different treatment steps and important design parameters of a conventional attached growth biological wastewater treatment plant.

2.7.1 Primary treatment

Preliminary treatment by screens or grit chambers is usually followed by primary sedimentation. The main objective of this treatment step is to remove a large fraction (50-70 %) of the total suspended solids in the wastewater. Since suspended solids also contribute to the content of BOD in the wastewater, one should expect 25-40 % of the total BOD to be removed in the process (Metcalf & Eddy 2004). When followed by biological treatment, the primary sedimentation step contributes to improved conditions by lowering the oxygen demand and the rate of energy consumption as a result of BOD removal. Removal of suspended solids also reduces the risk of operational problems in the next treatment processes.

The most important design parameter for a sedimentation basin is the retention time, which should be adequate for particles in the wastewater to flocculate and settle. Based on the average rate of wastewater flow, sedimentation basins are normally designed for a hydraulic retention time in the range of 1.5 to 2.5 hours (Metcalf & Eddy 2004). Lower values will lead to an insufficient removal of particles, while too long retention time will lead to a higher rate of break-up than forming of new aggregates, and thus a less efficient removal (Davis

2011). The retention time is determined as the product of the surface area and the depth of the tank. When considering the basin geometry, one should use a depth which provides the particles enough space to flocculate, but not so deep that the particles cannot reach the sludge layer at the bottom of the tank within the hydraulic retention time. A primary sedimentation tank is normally designed as a rectangular or circular basin, with at least two separate units, so that maintenance work can be carried out without closing down the plant or reducing the treatment efficiency.

The most important operation and maintenance practices for a sedimentation tank is the removal of solid sludge at a frequency high enough to avoid an interruption of the basins detention time, together with regularly cleaning / brushing of the basin walls.

Significant removal of pathogenic organisms is not expected in primary treatment, up to 1 log unit reduction could be expected (WHO 2006).

2.7.2 Secondary treatment

In general, biological wastewater treatment is based on the principle that microorganisms oxidize dissolved and particulate biodegradable matter into simple end products, which can be removed from the wastewater stream as sludge. Such processes can also remove suspended and non-settleable colloidal solids to a certain degree, as they are captured in biological flocs or biofilm. Nutrients such as nitrogen and phosphorus could also possibility be removed either as a part of the solids content or through biological decomposition (for nitrogen removal, see chapter 2.4.2.1). As an overview, the main purpose of secondary biological treatment is to remove readily biodegradable BOD that has escaped the primary treatment, in combination with further removal of suspended solids (Davis 2011). Biological treatment can be achieved either in the presence of oxygen (aerobic processes) or in the absence of oxygen (anaerobic processes). Two main types of biological treatment are common in wastewater treatment, one being suspended growth biological treatment, also known as activated sludge process, and the other being attached growth biological treatment, also known as biofilter process.

In a biofilter process, the principle of attached growth of biofilm in the presence of air on a filter media such as rock or plastic is practiced. Microbial activity will occur when untreated wastewater flows through the filter and is distributed on the surface of the filter media which will then be covered by biofilm. The biofilm, then containing biodegraded end products and suspended solids will grow thicker and thicker until it is released by sheer forces and distributed as flocs with the effluent, making room for new biofilm to occur. A typical biofilter design is the trickling filter. A trickling filter is typically shaped as a circular bed of filter media with a depth of 1-10 meter (depending on the weight of the filter media), where wastewater is evenly distributed on the surface through rotating arms set in motion either by an electric motor or by the dynamic reaction from the wastewater distribution. The trickling filter must be designed with an underdrain capable of leading away the wastewater flow and released solids without being clogged. The underdrain also serves as a ventilation channel where natural draft is used for aeration, and should be constructed so that the flow doesn't fill more than half of the channel, to allow air passage (Metcalf & Eddy 2004). An alternative to natural draft ventilation is the use of forced draft by fans, which has the benefit of a stable oxygen supply, but are more costly and energy demanding.

At optimized performance, reduction of pathogenic bacteria up to 2 log units can also be achieved in secondary treatment systems, depending on the suspended solids concentration (WHO 2006).

Nitrogen removal in biological treatment

To achieve nitrogen removal through biological treatment, the processes known as nitrification and then de-nitrification needs to occur. In nitrification, ammonia (NH_4-N) is oxidized to nitrite (NO_2-N) before nitrite is oxidized to nitrate (NO_3-N), both steps under the presence of oxygen. Each of the steps also depends on the presence of a specific group of autotrophic bacteria, respectively *Nitrosomonas* and *Nitrobacter*. The oxidation could possibly be carried through by other groups of bacteria, but the ones mentioned are the most common. The general formulas of the nitrification process are as follows (Metcalf & Eddy 2004):

Nitroso-bacteria:
$2NH_4$ (ammonia) $^+ + 3O_2 \rightarrow 2NO_2^-$ (nitrite) $+ 4H^+ + 2H_2O$

Nitro-bacteria:
$2NO_2^- + O_2 \rightarrow 2NO_3^-$ (nitrate) $+ 2H^+ + H_2O$

Total oxidation reaction:
$NH_4^+ + 2O_2 \rightarrow NO_3^- + 2H^+ + H_2O$

To achieve complete removal of nitrogen compounds, de-nitrification of nitrate needs to occur. This happens as nitrate is reduced to nitric oxide, nitrous oxide and then nitrogen gas which are natural occurring gas in the atmosphere. Groups of both heterotrophic and autotrophic bacteria are capable of de-nitrification with *Pseudomonas* being the most common type. Biological de-nitrification involves the biological oxidation of many organic substrates using nitrate or nitrite as electron acceptor instead of oxygen. The reduction steps occurring in de-nitrification are as follows (Metcalf & Eddy 2004):

NO_3^- (nitrate) $\rightarrow NO_2^-$ (nitrite) $\rightarrow$ NO (nitric oxide) $\rightarrow N_2O$ (nitrous oxide) $\rightarrow N_2$ (nitrogen gas)

In a biofilter process, nitrification can be achieved in addition to BOD removal at low organic loadings. Heterotrophic bacteria are more competitive than nitrifying bacteria, because of higher yield coefficients and faster growth rates. As a result, significant nitrification only occurs after the BOD concentration is reduced to a certain level. In secondary treatment units, nitrification is usually designed for in combination with BOD removal.

Secondary sedimentation tank

As the effluent from biological filters contains biological flocs to a large extent, there is a need of a second clarifier before further distribution of the wastewater. This is normally obtained by a secondary sedimentation tank where solids are settled and removed from the wastewater stream as sludge. Design criteria and maintenance and operation practices for a secondary sedimentation tank are similar to the criteria for a primary sedimentation tank.

2.7.3 Tertiary treatment

Tertiary treatment refers to a number of different treatment options that follows conventional secondary treatment. To accomplish nutrient removal sufficient to limit the risk of eutrophication of sensitive water bodies, an additional treatment step after secondary treatment is often necessary. These types of treatment steps, which also go under the term "advanced wastewater treatment" because of their generation of advanced techniques, could be designed in a variety of ways using different techniques (Davis 2011).

A typical facility for tertiary nitrogen removal is the use of filtration after secondary treatment. At this stage, the BOD concentration would normally be very low (<10 mg/L), providing a good basis for nitrification to occur. Important parameters at this stage are ammonia loading rate, oxygen availability, packing design and temperature (Metcalf & Eddy 2004). Normally there is limited oxygen availability in the upper portion of the biofilter, preventing nitrification.

Tertiary filtration is also effective with regard to pathogen removal, as pathogens are removed from the wastewater stream when passing through sand or other porous media. Reduction of bacteria up to 3 log units can be achieved in tertiary filtration (WHO 2006).

2.8 Analytical methods for WWTP performance assessment

Given the results of the treatment efficiency of a WWTP, it is desirable to transfer the data into an analytical context which allows for the expected treatment efficiency of different parameters specifically for the given treatment steps, in combination with the environmental guidelines of effluent discharge. The object of this action is to create an index which gives a assessment of the treatment plant as a whole. The expected treatment efficiency for specific treatment units can be found in various literature sources, while the maximum discharge guideline values are given by national standards (e. g EPA Ghana).

There are several possible ways to attack this challenge. Altayem Qasem describes a number of different condition rating models in his thesis "Performance Assessment Model for Wastewater Treatment Plants" (Qasem 2011). These include the statistical regression analysis technique, Artificial Neural Networks (ANN), the multi-attribute utility theory (MAUT), and the analytical hierarchy process (AHP). Below follows a short review of these.

2.8.1 The statistical regression analysis technique

Regression analysis is used to determine the relationships between dependent and independent variables based on statistical data. Regression is a generic term for all methods attempting to fit a model to observed data in order to *quantify the relationship* between two groups of variables. The fitted model may then be used either to merely *describe* the relationship between the two groups of variables, or to *predict* new values (www.camo.com).

2.8.2 Artificial Neural Networks (ANN)

An artificial neural network (ANN) is a computational model that approaches the structural functions of biological neural networks. The "neurons" of the models are constructed out of a large number of highly interconnected processing elements, which works in unison to solve given problems (Hamed et al. 2004). Through a mathematical imitation of human brain learning mechanisms, one can deal with dynamic and complex real-life systems. A recognized advantage of the ANN technology is that the structure of an ANN model can be changed during the analysis, as the neural network is a random function approximation tool that changes based on the input and the output of the model.

An example of a basic structure of a 3-layer feed-forward ANN model with four separate inputs is given in figure 2. The sum of the network's number of layers, the number of neurons in each layer, the activation function of each layer and how the layers connect to each other make up the networks architecture. The structure of a basic feed-forward ANN model could consist of one input layer, one hidden layer and one output layer which are connected but have no feedback connections. Further on, the weighted sum of input values are transferred to the neurons in the hidden layer, where they are transformed with the aid of an activation function. The output of the hidden layer then functions as input for the output layer, where it is again transformed. By introducing *network training*, one can minimize the error function by searching for a set of connection strengths and biases that causes the ANN to produce outputs that are equal or close to predefined targets (Hamed et al. 2004).

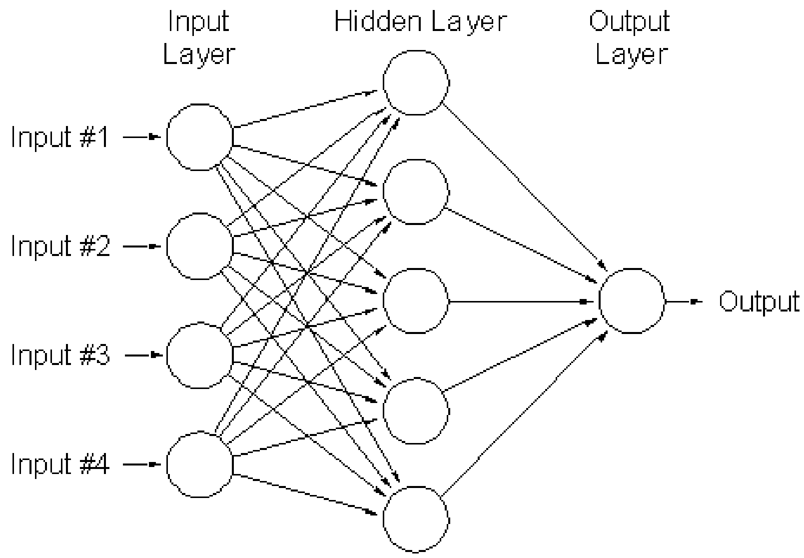

Figure 2: A representation of a simple 3-layer feed-forward artificial neural network with four inputs, 5 hidden nodes, and one output (SMIG 2002)

The ANN technique has been used in many studies as a predictive tool in water and wastewater applications. Hamed et al. (2004) shaped an ANN model for prediction of the performance of a major conventional wastewater treatment plant in Cairo, Egypt, using BOD and SS values at different stages of the plant as the input data. Daily records of the BOD and SS concentrations were obtained through a period of 10 months. The pair of BOD and SS data sets was then divided into separate groups, each containing both *training* and *testing* data. The suitable architecture of the model was determined from making trails and finding which setup gave the minimal error term in both training and testing data. The work provided a good tool for BOD and SS prediction, still highlighting the possibility of making the model stronger by including additional parameters such as pH and temperature. Mjalli et al. (2006) developed an ANN model for predicting the performance of a wastewater treatment plant in Doha, Qatar, using BOD, COD and SS as input parameters. The model was tested for different historical input-output data collected from various locations at the plant. The input-output was then grouped into two vectors and then subdivided into three groups: training, validation and testing. The full model architecture was then determined by testing. The results showed that a multi-input approach gave reasonable results for predictions of the plant performance. It was also stated that an ANN modeling approach was a good way of dealing with a plants high nonlinearity and the non-uniformity and variability of the crude supply as well as the nature of biological treatment.

2.8.3 The Multi-attribute Utility Theory (MAUT)

The multi-attribute utility theory (MAUT) functions by subdividing or breaking down problems into sublevels. By combining different single attributes into an aggregate function the attributes of each alternative are evaluated accordingly and the overall evaluation of an option is achieved (Qasem 2011). The different attributes within an alternative needs to be measured with a numerical value within a 0-1 scale with 0 representing the worst performance and 1 the best. Setting up a MAUT model is a three-step procedure. First, one has to describe the problem through a multi-attribute utility function. Then the weight of each of the utility functions must be determined, based on the importance of each of these utility functions. The last step is to create a single utility index for all the alternatives (Qasem 2011).

2.8.4 The Analytical Hierarchy Process (AHP)

The analytical hierarchy process is a model used to evaluate different decision alternatives by introducing quantitative and qualitative factors. The method has the ability to provide an overall rating out of an aggregate of alternatives, by rating relative weights of the different alternatives. Thus, it is a suitable tool for analyzing complex decisions. An AHP model builds on a hierarchical structure of a problem, which represents the relationship between goal, criteria and sub-criteria. The structure of an AHP model is shown graphically in figure 3.

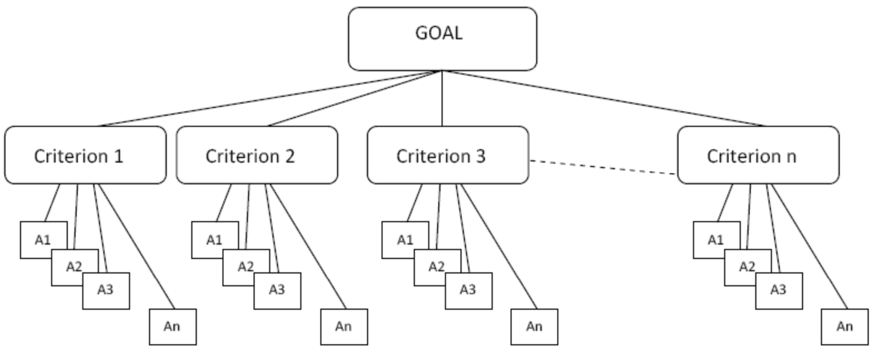

Figure 3: AHP Structure (Qasem 2011)

Within each hierarchical level the weights of different elements are determined separately. The decision on the final goal is determined as a result of all the weights of the different alternatives. Comparison matrixes are used to do the evaluation between the alternatives, by comparing two at a time, with respect to the impact on an element above them in the hierarchy. The evaluation of each alternative is transferred to as numerical value, which are in the end are calculated for each of the alternatives and compared to each other based on the numerical value.

Qasem (2011) created a performance assessment tool for managing WWTPs as a combined AHP-MAUT model. SS and BOD removal efficiencies were used as indicators to measure the treatment performance for the primary treatment phase. For the secondary treatment phase (activated sludge process), BOD, sludge volume index (SVI) and mixed liquor volatile suspended solids (MLVSS) concentrations were employed. To evaluate the performance of the tertiary treatment phase, CFU tests and the production of harmful disinfection byproducts (DBFP) were used. Each treatment phase was given relative weights based on their impact on the overall treatment performance. As a recommendation for further enhancement, Qasem suggested to include ANN models to show the relationship between different factors.

2.9 Summary of literature review

The main findings of the literature review can be summarized as follows:

1) Developing countries experience a huge gap in the coverage of collection and treatment of domestic wastewater. This result in a high quantity of pollutants discharged into natural water bodies, creating a negative impact on both the environment and the sanitary conditions.

2) The first step of upgrading a treatment plant would be to assess the present performance. This can be achieved by creating a computer model for the analysis of the activity of critical parameters throughout the plant, such as BOD and SS concentrations. Potential tools for this assessment include regression analysis, MAUT, AHP or ANN models.

3.0 Methodology

3.1 Description of the study area

The KNUST WWTP is located in Kumasi, the second largest city of Ghana. Kumasi has a total population of close to 2 million. The city is located in the southern central part of Ghana and is the capital of Ashanti region. The KNUST WWTP covers the campus area of KNUST (Kwame Nkrumah University of Science and Technology), which in 2009 was estimated to have a population of about 25,000 (Fosu 2009). An accurate estimation of the total population is difficult to determine, since it is known that many students unofficially lives in the student halls.

3.1.1 Climate

The climate of Kumasi is categorized as tropical wet and dry, with relatively constant temperatures throughout the year. The average minimum temperature ranges from 21° C (August-September) to 23° C (February-March). The average maximum temperature varies from 27° C (August) to 34° C (February) (climate-zone.com 2012). Kumasi receives an average 1488 mm of precipitation annually, with the main share appearing in the rainy season from March through July. A second, shorter rainy season appears from September to November. The dry season is experienced from December to February, as a result of the dry and dusty West African trade wind blowing from Sahara into the Gulf of Guinea, known as the harmattan. The mean relative humidity yearly is recorded as 83.2 %, with a monthly variation from 75% in February to 87 % in June – October (climatemp.info 2011).

3.2 Description of the KNUST WWTP

3.2.1 Background

The KNUST WWTP started to operate in 1964, after a construction period of three years. At this time, the plant received and treated wastewater from about 700 students living in the KNUST campus area. Data of the initial design capacity is missing, but it is known from the project drawings that a future extension of the plant was planned. This is yet to be implemented. From 1964 until now, the student population connected to the plant has increased drastically. The plant was operating from 1964 until it broke down in 1995. From 2001 to 2007 the plant was under rehabilitation, and until it was operational again in 2007 the wastewater was discharged to the nearby stream without treatment (IWMI 2008).

3.2.2 Characteristics of catchment area and influent wastewater

The KNUST WWTP receives wastewater from a large share of the facilities within the KNUST campus area, including all the student halls, most of the campus hostels, most of the university's faculties and research units, the staff bungalows and some administrative units such as the main library, the Great hall and the main administration building. The total population connected to the plant was in 2009 estimated to be around 25000.

The main fraction of the wastewater treated by the KNUST WWTP is thus typical domestic wastewater coming from bathroom and kitchen sources. There are no large-scale industries or stormwater connection systems within the catchment area of the plant.

3.2.3 Technical details

All the wastewater from the student's residence halls flows by gravity, while the wastewater from the faculty area is pumped, until it all reaches the main pumping station. At the inlet of the pumping station, the wastewater goes through a screening chamber for large objects and particles to be removed. From this point it is pumped further on to a primary sedimentation tank, where it further flows by gravity via a dosing chamber, through a biological trickling filter unit, a humus tank and a tertiary filter unit before it is discharged as effluent water into the nearby stream. As of the time of this study, the dosing chamber was not used for chemical dosing, but functioned as an additional settling tank. The trickling filters are designed in two parallel lines, and the tertiary filters consist of three parallel basins. Sludge collection is implemented from the following units: the sedimentation tank, the dosing chamber and the humus tank. The sludge is transported to a separate unit of ten parallel drying beds. The backwashed water and percolated water from the drying beds are both led back to the primary sedimentation basin and goes with the sludge to the drying beds. A flowchart of the plant is given in figure 4.

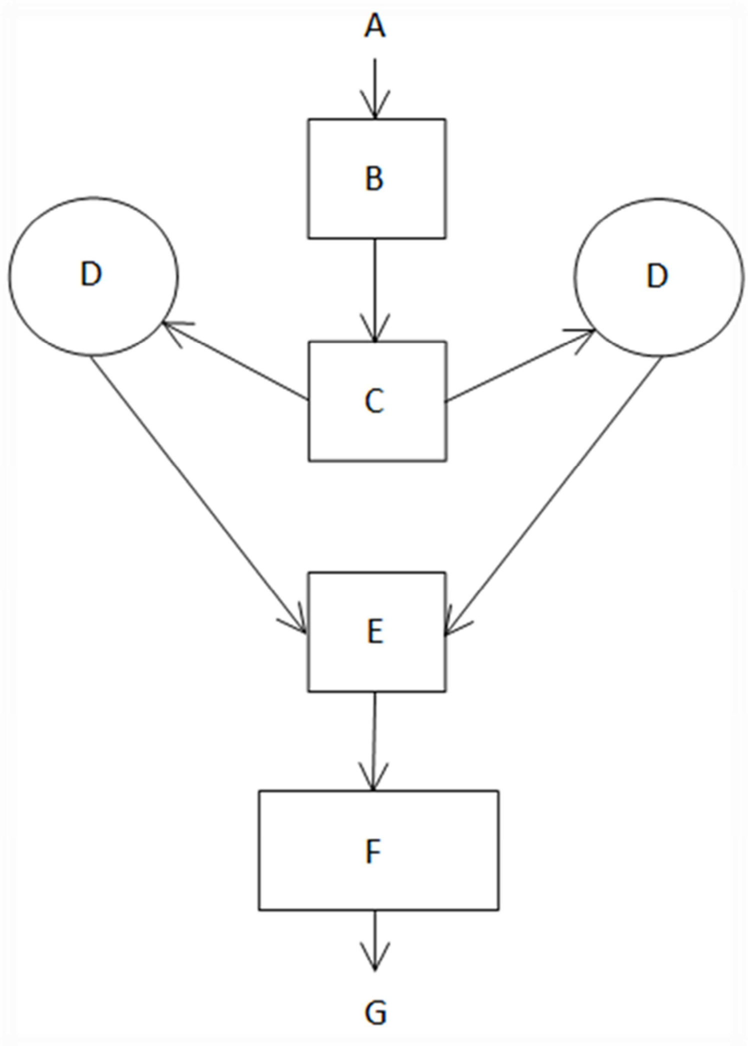

Figure 4: Flowchart of the KNUST WWTP: A) Influent, B) Primary sedimentation tank, C) Dosing chamber, D) Trickling filters, E) Secondary sedimentation tank, F) Tertiary filters, G) Effluent

The existing main treatments units are described as detailed as possible in the following section.

Primary sedimentation tank

The Primary sedimentation unit is designed as two parallel basins (Figure 6 and 7) with a joint inlet (Figure 4) and a joint outlet (Figure 8).

Figure 5: Inlet of primary sedimentation tank

Figure 6: Primary sedimentation tank

Figure 7: Primary sedimentation tank

Figure 8: Outlet of primary sedimentation tank

Dosing chamber

From the primary sedimentation tank, the wastewater flows by gravity to the dosing chamber. The dosing chamber only functions as a sedimentation tank, as it is desludged, but not used for chemical dosing. From the dosing chamber the wastewater is distributed further to the trickling filters via two parallel arms, with the aid of a syphon.

Trickling filters

The trickling filter unit is the main treatment step of the plant. It is designed as two identical circular filters, with diameters of approximately 16.0 m and depths of approximately 1.35 m (Figure 7 and 9). Rock packaging is used throughout the whole media. The wastewater is distributed through a pivot in the middle of the filter basin (Figure 10), and each filter has four rotating arms with 18 nozzles on each arm. The arms are driven solely by hydraulic forces.

Figure 9: Trickling filter

Figure 10: Pivot of trickling filter

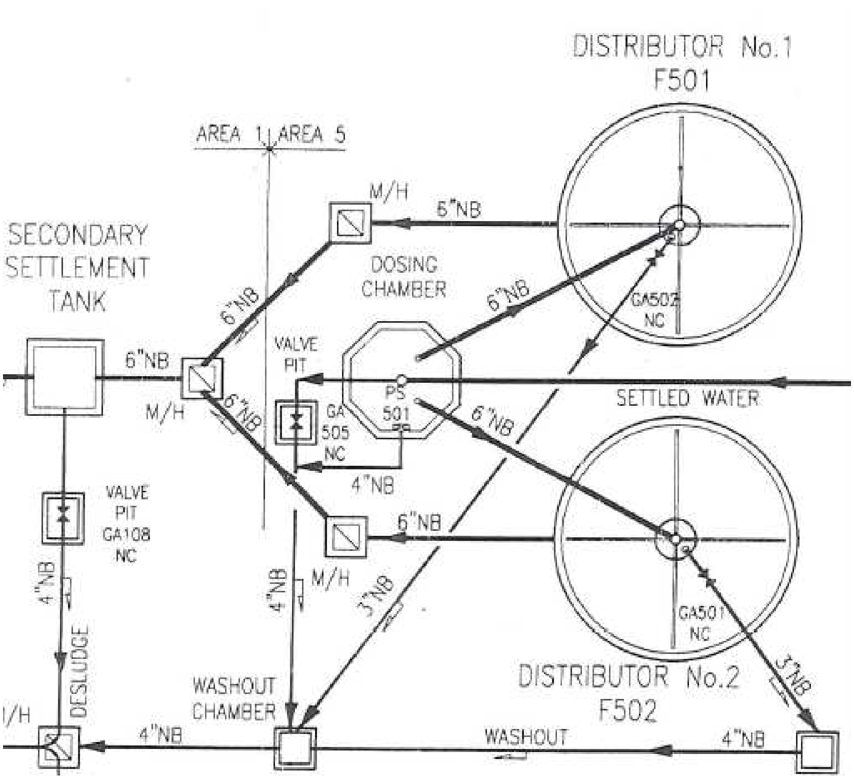

Figure 11: Project drawing of trickling filter units

Secondary sedimentation tank

The effluent of the trickling filters is further distributed to the secondary settlement tank (Figure 12), which has a desludge capacity. The wastewater is distributed into the middle of the basin, from where it flows outwards to the outer edge. The basin is surrounded by a threshold leaving the sedimented materials behind for sludge outtake. A gutter covering the basins distributes the treated wastewater to the outlet, and into the inlet of the tertiary filters.

Figure 12: Secondary sedimentation tank

Tertiary filters

The tertiary filter unit consists of three parallel basins. As of the time of the study, only two of the basins were under operation as the last one is not filled with filter media. As filter packaging, rocks in three different degrees of coarseness is used, with the most course layer as the top layer and the finest at the bottom. Each basin is constructed with an underdrain for backwashing, and there is a joint washout for the backwashing water to be pumped back to the primary sedimentation tank. The effluent of the tertiary filters is distributed by a pipe to the discharge point at the Wiwi River.

Figure 13: Tertiary filters

Figure 14: Filled tertiary treatment basin

Figure 15: Empty tertiary treatment basin

Sludge drying beds

The sludge drying beds receives sludge from the primary sedimentation tank, the dosing chamber and the humus tank. The beds are constructed as 10 identical sized basins, each holding a surface area of approximately 9.0 x 2.1 m. The sludge from all the sedimentation tanks is pumped to the drying beds via the sludge pumping station. It consists of two pumps, for one to run while the other one is on standby. The sludge pumping station is also used for pumping the percolated water from the drying beds back to the primary sedimentation tank, together with the backwashed water from the tertiary filters. The sludge is usually dried in the beds for a period of 3-4 weeks. In rainy periods, the drying period is usually longer. After this period, the dried sludge is removed manually and can be used for agricultural purposes.

Figure 16: Sludge drying beds

Figure 17: Single sludge drying bed

3.2.4 Operation and maintenance procedures

The main actions regularly undertaken at the plant is as follows:

- Backwashing of tertiary filters are scheduled for every shift. The backwashed water is pumped back to the primary sedimentation tank. As of the time of the study, this action is normally undertaken just 1-2 times per day when the plant is running;
- Desludging of the primary sedimentation tank, the dosing chamber and the humus tank respectively;
- Emptying of the sludge drying beds when required.

Other action that is required, but not undertaken at a regular basis includes:

- Cleaning of the primary sedimentation tank, the dosing chamber and the humus tank, with the purpose of removing sludge stuck to the walls. Due to lack of adequate brushes, this is not done at the moment;
- Cleaning of trickling filter media. According to the plant manager, this action has not been undertaken since the plant started to operate;
- Major maintenance for complete cleaning of the treatment units and to change destroyed or worn-out parts. According to the plant manager, this should be done twice a year, but due to lack of funding it is not carried through at this frequency.

3.2.5 Personnel

The KNUST WWTP has a total of 13 workers:

- 1 plant manager;
- 6 permanent workers;
- 6 casual workers.

There are workers on the plant every hour of the week, divided on three daily shifts. The morning shift (6 a.m. -2 p.m.) demands two permanent workers and six casual workers, while the evenings shift (2 p.m. – 10 p.m.) and the nights shift (10 p.m. – 6 a.m.) demands two permanent workers on each of them. During the weekend, there are only two workers on every shift.

During the mornings shift, two workers are stationed in the control room at the main pumping station. These undertake the cleaning of the screening area and monitoring of the main sewer line. Two workers are also stationed at the main plant, carrying out cleaning of the different treatment units. This include backwashing of the tertiary filters, which are in the principle undertaken every shift.

Out of today's stab, only the plant manager has finished relevant education, as he a graduated technician. Out of the permanent staff, two have other education and two are non-educated. The non-permanent staff includes one worker with a diploma in mechanical engineering, 3 with other degrees and two are non-educated. Only the permanent staff has gone through specific training in operating the plant.

3.2.6 Financial situation

A full, detailed overview of the plant's financial situation is not accessible, though some of the general data are achievable. The treatment plant is funded by the university, with salaries to the workers and general operation costs as the main expenses. Funding for maintenance is not adequate. According to the plant manager, spending money on the treatment plant is not a high priority as it doesn't generate any income to the university.

As today, the monthly salaries paid to the workers are as follows (GHC = Ghanaian Cedi):

- 1 plant manager – 690 GHC
- 3 senior permanent workers – 800 GHC
- 3 laborers – 650 GHC
- 6 casual workers – 113.85 GHC

This gives a total monthly wage expense at 2253.85 GHC. Given an exchange rate to EURO on 0.42 it equals approximately 950 EURO.

For general maintenance costs, an approximation given by the plant manager gives says that 500 GHC should be provided monthly for expenses on grease and brushes. In addition, a biannual amount of 1500-2000 GHC should be provided for major maintenance purposes.
In addition to this, there are economical costs associated with the general running of the plant, mainly in relation to the running of the pumping stations.

3.3 Sampling procedure

The setting up of a proper sampling program was the first step into characterizing the constituents of the wastewater and thereby documenting the performance of the treatment plant. Several factors were considered to meet the need of a representative sampling program. These factors included number of and selection of sampling locations, type of samples (grab or composite samples), sample sizes, time intervals between samples and total number of samples needed to achieve statistically representative output values from the analyses. The details of the sampling program is listed as follows:

3.3.1 Sampling points

The selection of critical control points throughout the treatment plant was designed to achieve a better overview of the plant's performance as a whole, together with a detailed picture of the performance of the different treatment steps. The control points, which served as locations for the samples, was as follows:

A) Inlet of the treatment plant - Samples were taken at the main pumping station, in the gutter between the screening unit and the pump house.

B) Inlet of the biofilters – Samples were taken directly from the nozzles on the rotating arms of the trickling filters. Since there are two parallel filter units, the sampling was alternated, starting on sampling day 1 at the filter unit corresponding with outlet C1, and then alternating every sampling day.

C1) Outlet of biofilter 1 – Samples were taken from the gutter available at the effluent chamber corresponding to biofilter 1.

C2) Outlet of biofilter 2 - Samples were taken from the gutter available at the effluent chamber corresponding to biofilter 2.

D) Effluent of secondary sedimentation tank – Samples were taken from the gutter surrounding the sedimentation unit.

E) Effluent of tertiary filters – Samples were taken from the joint outlet of the three filter units.

3.3.2 Type and size of samples

Each sample was implemented as a composite sample out of three grab samples. For each control point there was collected a minimum of 1 litrer for the chemical/physical analyses and a total of 0.25 litres for the microbial analyses. The amounts are correlated to the actual minimum amount needed to carry out the appurtenant analyses. For the microbial analyses, sterile plastic bottles were used, while for physical / chemical and helminth analyses cleaned water bottles were used. After collection, the samples were immediately transferred into a cooling box and transported to the laboratory for analysis.

3.3.3 Scope of sampling regime

Sampling was carried out from 07.02.2012 to 01.03.2012. Sampling was accomplished every day of the week except Sundays. Due to frequent interim closedowns of the plant, in addition to a sickness period, sampling was not undertaken every day through the period. The total number of sampling days was 15. Sampling was carried out from 8:00 and 12:00 each morning during the sampling period.

3.4 On-site measurements

Parameters such as Dissolved oxygen (DO), temperature and pH of the wastewater at all the sampling points were measured onsite. The readings were done from the bottles for physical/chemical analyses immediately after sampling.

3.4.1 DO

The readings of the dissolved oxygen in the wastewater samples were performed using a handheld -YSI 550A dissolved oxygen meter. The sensor was cleaned with distilled water between every measurement.

3.4.2 Temperature

The temperature (represented in °C) was read with the YSI 550A meter, simultaneously with the D.O.-readings.

3.4.3 PH

The pH readings were undertaken using a handheld 'pH 1000 H, pHenomenal' pH-meter. The sensor was held in the sample until the pH-value was stabilized within a one decimal range. Between every reading, the sensor was cleaned with distilled water. The pH-meter was calibrated before the first sampling day, and controlled every day before sampling by taking reading from a buffer solution with a known pH-value.

3.5 Procedures for laboratory analyses

3.5.1 E. coli / TFC

The contents of E. coli and Total Fecal Coliforms in every sample were quantified using the Idexx Quanti-Tray method, following the procedure described below.

1) Dilution series of the raw wastewater samples was prepared. When the desired dilution was obtained, 10 ml of the selected dilution was pipetted into an Erlenmeyer flask containing 90 ml of distilled water. Then the solution was suspended with one snap pack of Colisure reagent (Figure 16) and shaken thoroughly and rested for some time until there were no large particles left.

2) When the reagent was completely mixed in the solution it was poured into a Quanti-Tray and sealed using the Quanti-Tray sealer. The sealed plates were then incubated at 35 °C for 24 hours.

3) After the incubation, enumeration of TFC and E. coli was performed. TFC was found by directly counting wells in the Quanti-tray with a red or magenta appearance (Figure 17). E. coli was detected by keeping the tray under a 6 watt, 365 nm UV light and counting the number of red/magenta or fluorescent appearing wells (Figure 18).

4) The enumeration was completed using the MPN table to quantify the level of TFC and E. coli per 100 ml of sample.

5) The enumeration was completed by implementing the number of positive wells in the MPN table, getting the quantity of bacteria per 100 ml of sample.

Figure 18: Colisure reagent

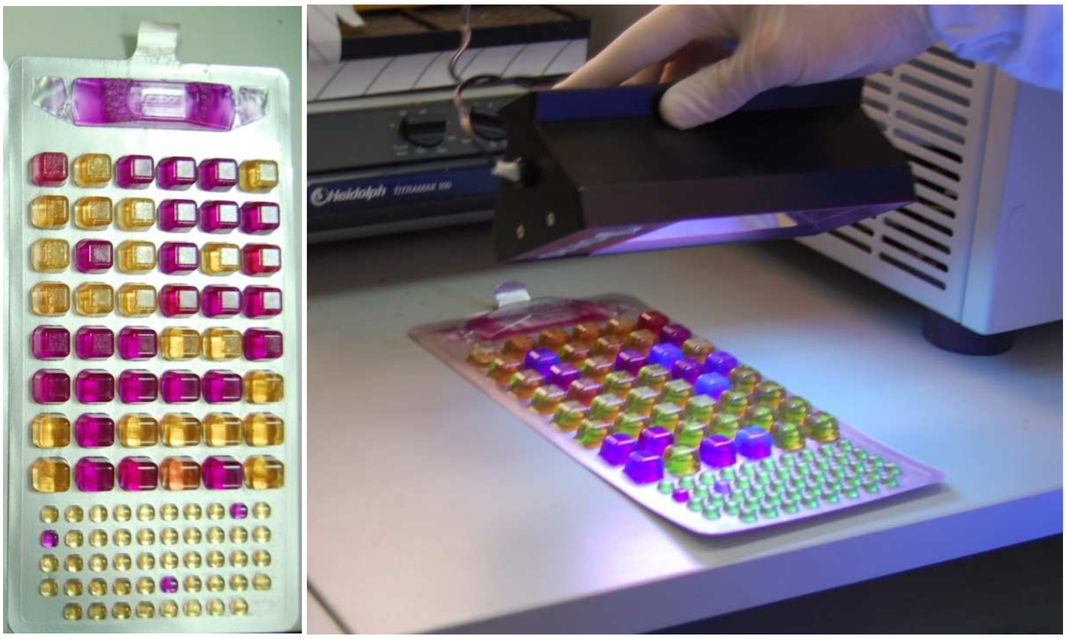

Figure 19: a) Incubated Quanty-tray, b) Incubated Quanti-tray inspected under UV-light

3.5.2 BOD_5

1) 1 ml each of phosphate buffer, magnesium sulphate, calcium chloride and ferric chloride solution was added to a 1 L volumetric flask. De-ionized water was added up to 1 L.

2) 1 ml of wastewater sample was added to a 500 ml beaker, and then filled up to 300 ml with dilution water. The pH was adjusted to a value within the range of 6.8-7.5 by adding acid or alkali.

3) 300 ml of dilution water was also prepared as control. Both the prepared samples and control bottles were then put into 300 ml incubator bottles. The D.O. for each sample was measured using a D.O. meter, before they were incubated in a BOD incubator for 5 days at 20 °C. The D.O. value was again measured after 5 days.

4) The calculation for BOD content in mg/l were done by the following formula:

$$\mathrm{BOD_5} = \frac{\mathrm{D_1} - \mathrm{D_2}}{\mathrm{P}}$$

The parameters in the formula represent the following:

D_1 = D.O. value in initial sample
D_2 = D.O. value in final sample
P = decimal volumetric fraction of sample used (ml of sample/300 ml)

3.5.3 Ammonia

The ammonia content in the wastewater was detected using the Wagtech Ammonia Test. The method is based on the principal that a green-blue indophenols complex is formed when ammonia reacts with alkaline salicylate in the presence of chlorine. The intensity of the color produced is proportional to the ammonia concentration. The procedure is described in the following section.

1) 10 ml of wastewater sample was filled into a test tube. One Ammonia No. 1 tablet and one Ammonia No. 2 tablet were added and mixed into the sample.

2) The sample was left standing for ten minutes for colour to develop, before a photometer reading was performed on wavelength 640 nm.

3) The amount of ammonia in the sample was then determined by reading the transmittance percentage from the photometer (Figure 20) and consulting the value into an ammonia calibration chart to get the ammonia concentration given in mg/l.

Figure 20: Wagtech photometer

3.5.4 Nitrate

The amount of nitrate in the wastewater samples was found with the aid of the Wagtech Nitratest. In this test nitrate is first reduced to nitrite, before the resulting nitrite is determined by a diazonium reaction to form a reddish dye. By using a zink-based Nitratest powder and a Nitratest tablet, the reduction stage is carried out.

1) 1 ml of sample was added to a Nitratest tube, by using a pipette. The Nitratest tube was then filled up to 20 ml with de-ionized water.

2) One Nitricol tablet was added and dissolved in the solution. The colour intensity was then read on a Wagtech Photometer (Figure 20) at wavelength 570 nm. Then the transmission percentage given from the photometer was consulted into the chart. The given value was then multiplied by 20 to get the nitrate concentration in the original sample in mg/l.

3.6 Flow readings

It was of great interest to generate an overview of the influent flow of the treatment plant, both in terms of total quantity and variations through the day and through the week.

This was achieved by manual readings from the flow-meter in the control room at the main pumping station of the treatment plant, performed by the casual workers at the plant. Data was collected at intervals of fifteen minutes from 6 a.m. to 12 p.m. The readings were carried out during a period of 9 days (09.02.12 - 17.02.12). This period included a total of seven weekdays during the period when the school was in session, in addition to one Saturday and one Sunday. Due to frequent power failures, there were breaks in the running of the plant which resulted in data gaps.

The collected data was summarized and generated graphically using Excel. One graph was generated for the Monday – Friday readings (Figure 19) and one was made for the Saturday – Sunday readings (Figure 20). The flow value connected to each time interval was determined as the average value of all daily readings for that given time interval.

3.7 Statistical review of data

All raw data from the microbial and chemical/physical analyses were entered into Microsoft Excel, finding mean values. All descriptive analysis and regression analysis were conducted using the statistical software Minitab 16.

4.0 Results

4.1 Introduction

The following chapter gives a descriptive analysis of the flow properties and the water quality of the KNUST wastewater treatment plant as measured during this study. The flow readings represent sampling point A only, while all other parameters are represented with values for sampling points A-E. All raw data are included in the appendix.

4.2 Physical-chemical parameters

4.2.1 Flow

The average flow throughout the week was calculated to be 895 m^3/ d, based on the observations. For weekdays, the average flow was 867.2 m^3/ d, while in weekends, the readings gave an average of 978.5 m^3/ d. The observed flow at the inlet of the plant shows a tendency of being at the highest in the morning around 7-8 AM, with an additional lower top in the afternoon around 7 PM in the weekdays. The weekend readings gave a curve with less distinct tops and lows. This corresponds well with the expected patterns of wastewater loadings. The flow characteristics for weekdays and weekends are given in figure 21 and 22.

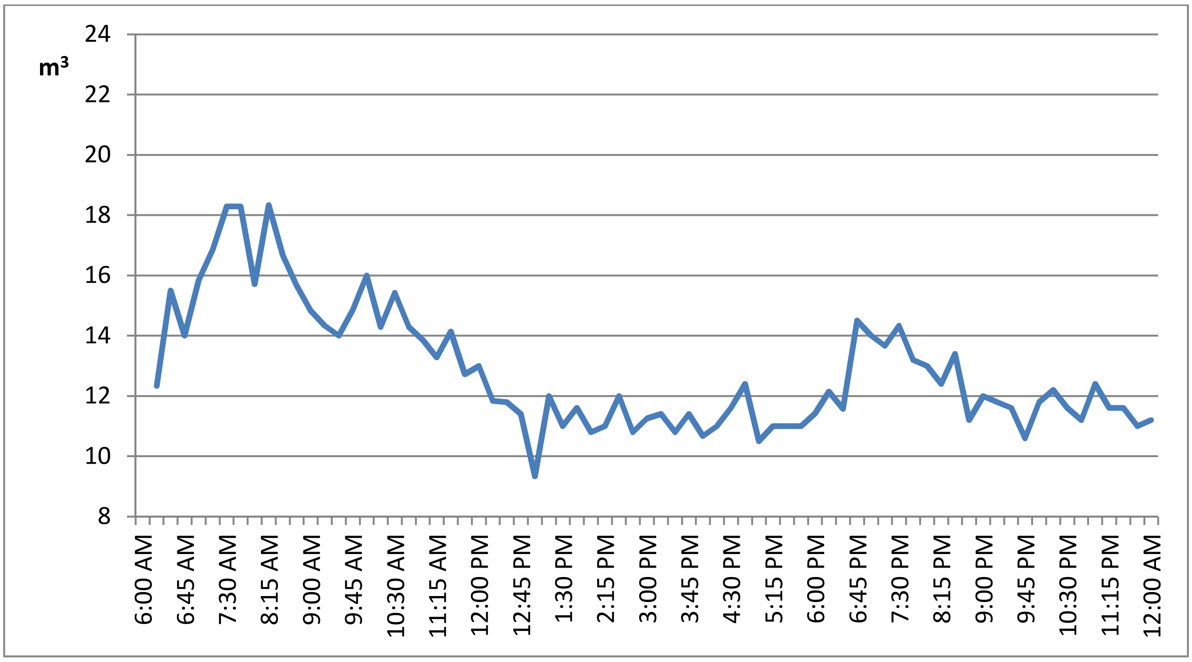

Figure 21: Flowchart Monday – Friday

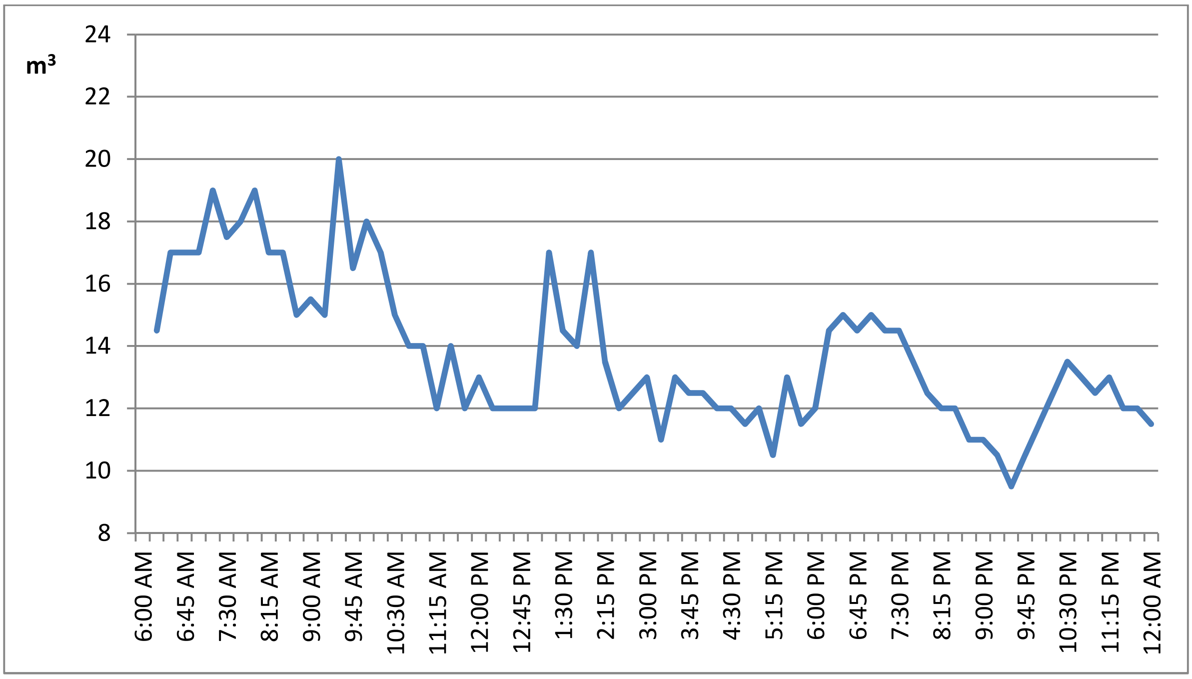

Figure 22: Flowchart weekend

4.2.2 Temperature

The variations in temperature are given in table 2 and figure 23. The influent to the treatment plant held an average temperature of 27.7 ° C during the sampling period. A slight decrease of temperature was observed in the inlet and the outlet of the biofilters, before the temperature increased again in the secondary sedimentation tank and the tertiary filters. The effluent held an only slightly lower temperature than the influent.

Table 2: Mean value (° C), standard deviation and range for temperature measurements at sampling points A-E

SAMPLING POINT	N	MEAN	STANDARD DEVIATION	RANGE	
				MIN.	MAX.
A	15	27.75	± 0.78	26.50	29.80
B	15	27.35	± 0.49	26.40	28.30
C1	15	26.97	± 1.52	23.80	29.10
C2	15	27.40	± 1.26	25.20	29.10
D	15	27.28	± 1.33	25.10	29.00
E	15	27.47	± 1.08	25.80	30.40

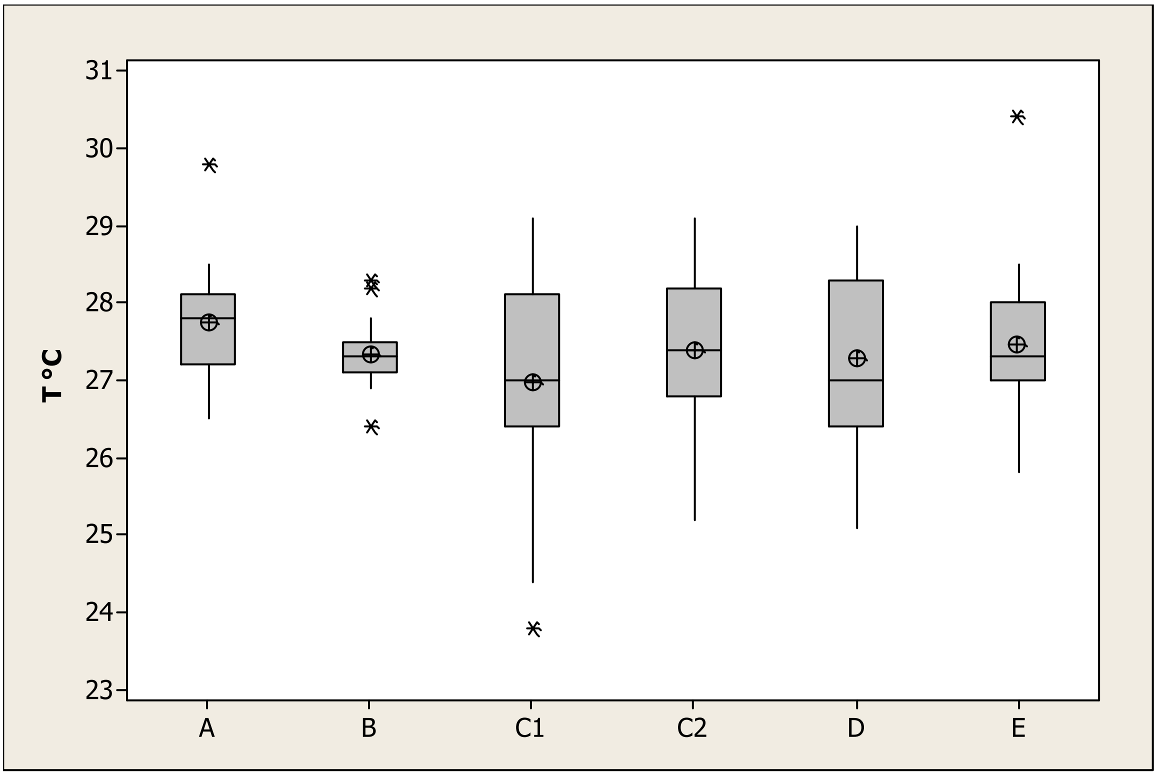

Figure 23: Temperature variation in the treatment steps.

4.2.3 DO

The changes in DO values through the treatment plant are presented in table 3 and figure 24. The readings indicate that the inlet water to the KNUST treatment plant held a DO concentration of 0.39 mg/l. A pronounced rise in DO concentration was observed in the outlet water from the biofilters, before the concentration again reached a value near the inlet value.

Table 3: Mean values (mg/l), standard deviation and range for DO values at sampling points A-E

SAMPLING POINT	N	MEAN	STANDARD DEVIATION	RANGE	
				MIN.	MAX.
A	15	0.39	± 0.24	0.11	0.91
B	15	0.35	± 0.28	0.09	1.00
C1	15	5.11	± 1.58	1.30	7.32
C2	15	5.20	± 1.45	2.59	7.32
D	15	1.92	± 1.50	0.12	5.25
E	15	0.45	± 0.32	0.11	1.18

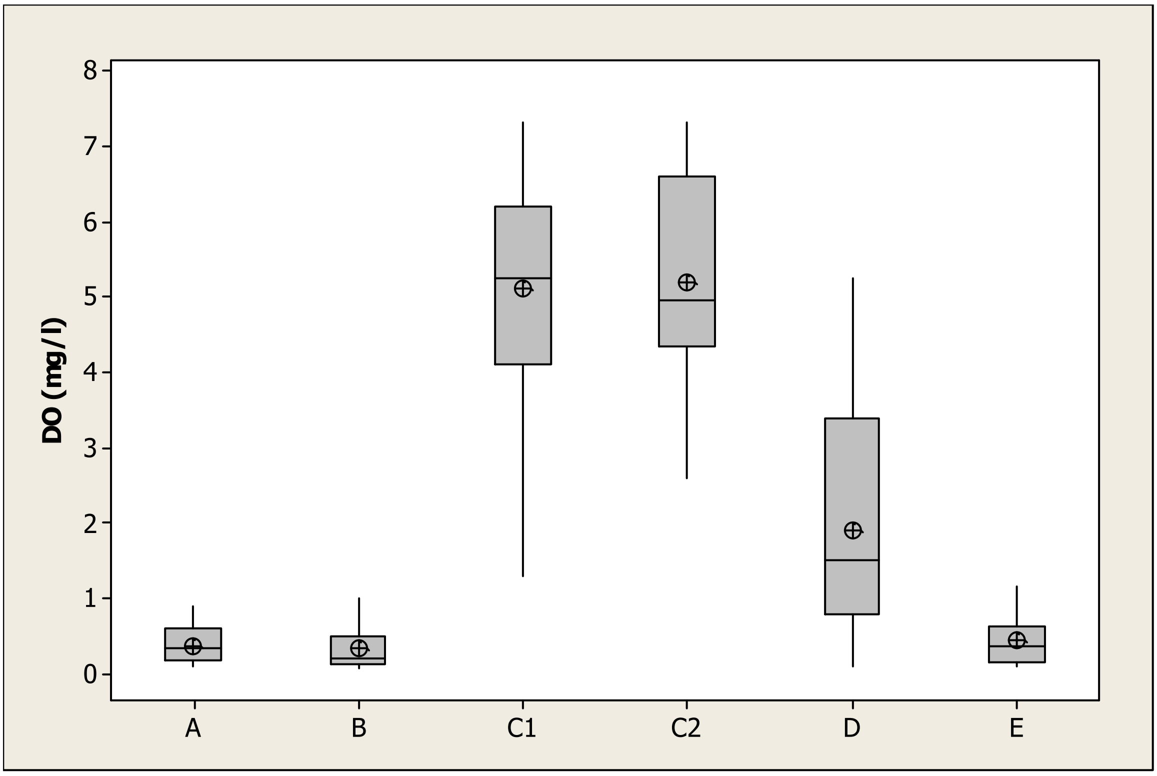

Figure 24: DO values in the treatment steps

4.2.4 pH

Variations in pH are described in table 4 and figure 25. The pH value was measured to have an average of 8.39 at the inlet of the treatment plant. At the inlet of the biofilters this value was decreased to 7.68, and kept close to stable through the remaining treatment units.

Table 4: Mean value, standard deviation and range for temperature

analyses for sampling points A-E

SAMPLING POINT	N	MEAN	STANDARD DEVIATION	RANGE	
				MIN.	MAX.
A	15	8.39	± 0.20	7.85	8.67
B	15	7.68	± 0.56	6.42	8.45
C1	15	7.69	± 0.20	7.35	7.96
C2	15	7.71	± 0.20	7.30	7.99
D	15	7.62	± 0.21	7.20	7.94
E	15	7.60	± 0.22	7.17	7.91

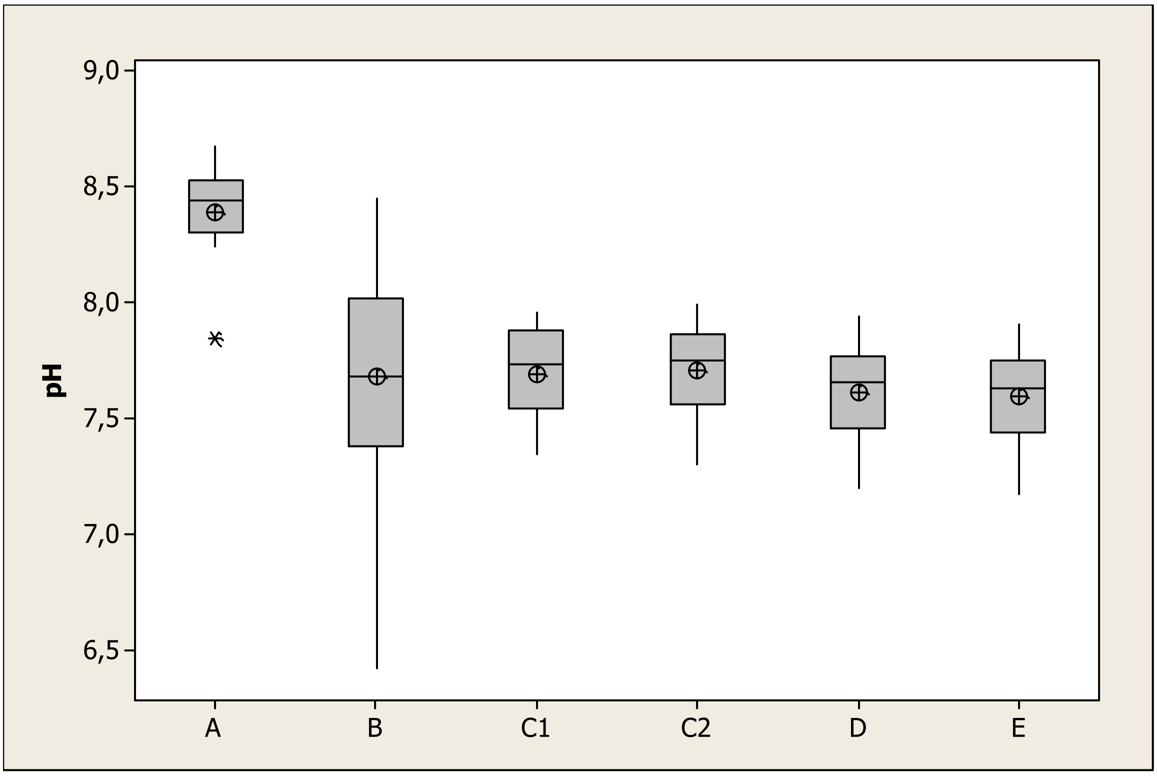

Figure 25: pH values in the treatment steps.

4.2.5 BOD_5

The concentration of BOD_5 at different levels of the treatment plant is presented in table 5 and figure 26, while figure 27 show the specific reduction rate at each treatment step. The treatment system performed a total reduction of 82.6 % of the influent BOD_5 concentration, resulting in an outlet value of 72.6 mg/l. The most significant decline of BOD_5 was found in the biofilters (67.2 % of initial concentration).

Table 5: Mean values, standard deviation and range for BOD_5

concentrations (mg/l) at sampling points A-E

SAMPLING POINT	N	MEAN	STANDARD DEVIATION	RANGE	
				MIN.	MAX.
A	14	417,79	± 131,26	220	670
B	15	378,53	± 102,70	204	590
C1	15	120,07	± 44,15	54	192
C2	15	113,00	± 43,31	48	180
D	15	103,00	± 51,36	48	210
E	15	72,60	± 37,13	30	150

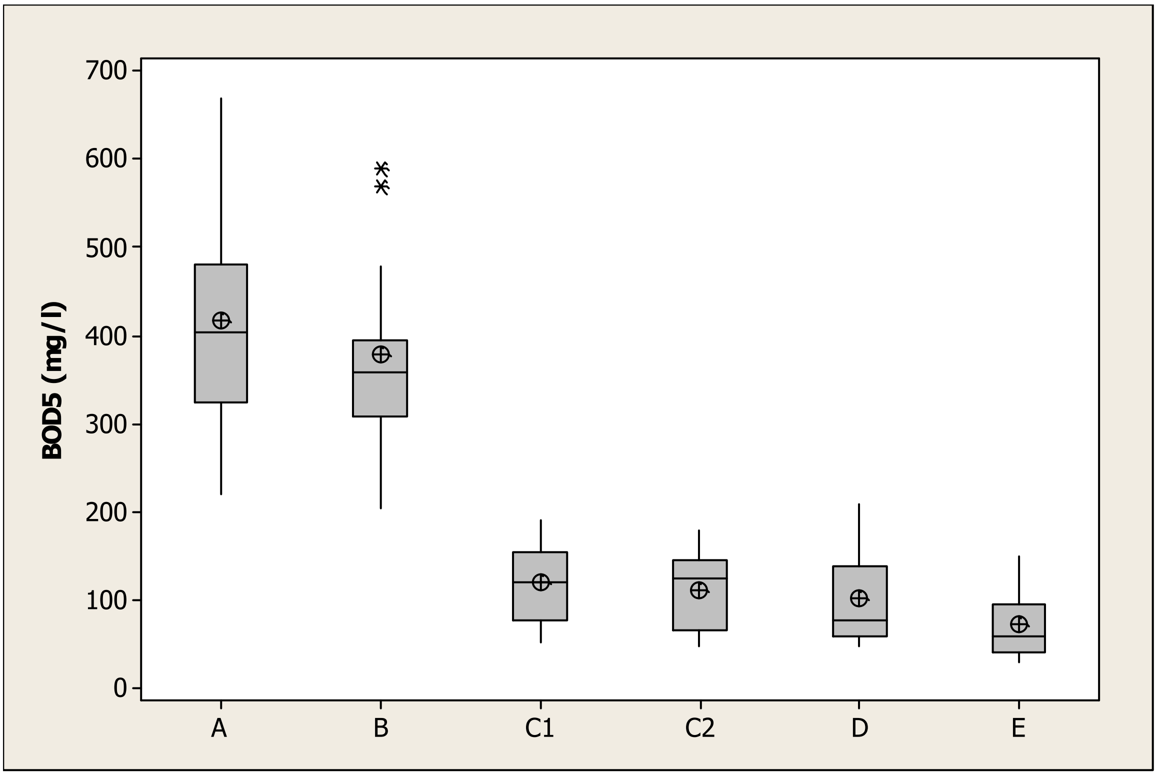

Figure 26: BOD_5 values in the treatment steps.

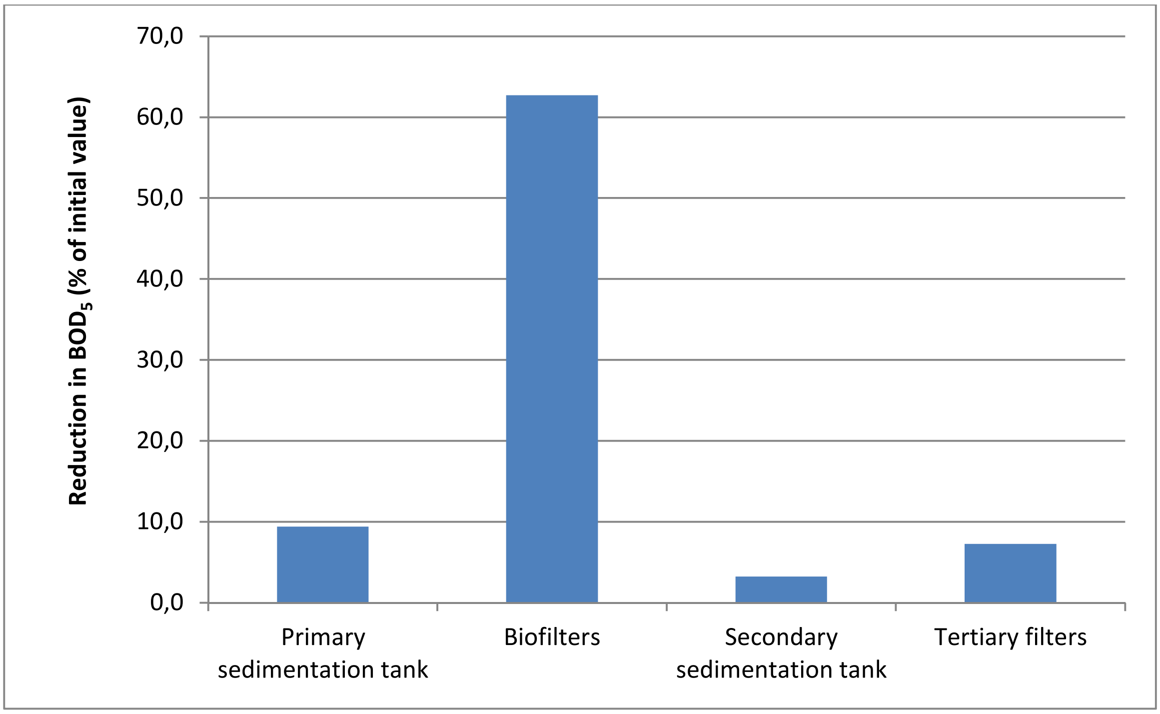

Figure 27: Reduction of initial value of BOD_5 in each treatment step

4.3 Chemical Parameters

4.3.1 Ammonia

The inlet held a concentration of 1.54 mg ammonia/l which was reduced to 0.26 mg/l at the point of the outlet (Table 6 and figure 28). As seen in figure 29, about 80 % of the initial concentration was reduced in the primary sedimentation tank and the biofilters, before the ammonia concentration held a rather stable state through the remaining treatment units.

Table 6: Mean values, standard deviation and range for total ammonia concentrations (mg/l) at sampling points A-E

SAMPLING POINT	N	MEAN	STANDARD DEVIATION	RANGE	
				MIN.	MAX.
A	14	1.54	± 1.10	0.56	4.22
B	15	0.79	± 0.60	0.28	2.11
C1	15	0.35	± 0.34	0.12	1.50
C2	15	0.29	± 0.27	0.07	1.19
D	15	0.29	± 0.45	0.04	1.87
E	14	0.26	± 0.25	0.02	0.92

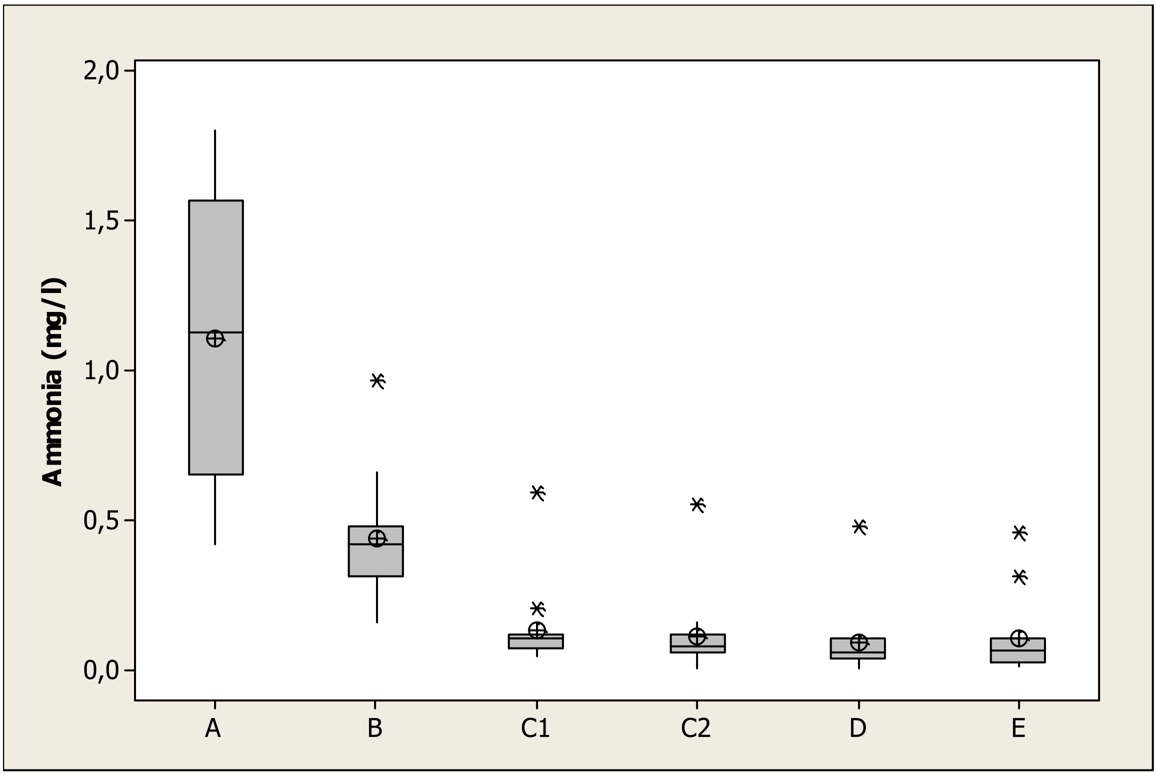

Figure 28: Ammonia values in the treatment steps

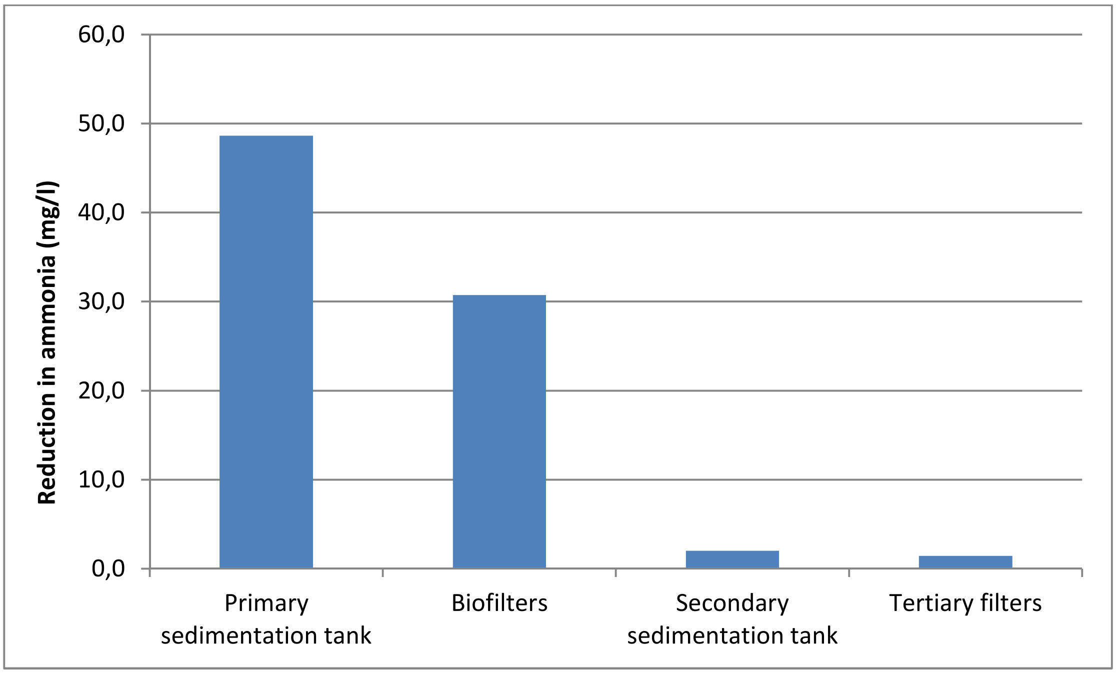

Figure 29: Reduction of initial value of ammonia in each treatment step

4.3.2 Nitrate

About 90 % of the inlet concentration of 1.1 mg nitrate/l was reduced during the tretmant steps. The main contribution was reduced in the primary sedimentation tank (about 60 %) and the biofilters (about 30 %). The full numbers are given in table 7, figure 30 and figure 31.

Table 7: Mean values, standard deviation and range for nitrate concentration (mg/l) at sampling points A-E

SAMPLING POINT	N	MEAN	STANDARD DEVIATION	RANGE	
				MIN.	MAX.
A	14	1.10	± 0.46	0.420	1.800
B	15	0.44	± 0.19	0.156	0.970
C1	15	0.13	± 0.13	0.048	0.590
C2	15	0.11	± 0.13	0.006	0.550
D	15	0.09	± 0.11	0.006	0.480
E	14	0.11	± 0.13	0.010	0.460

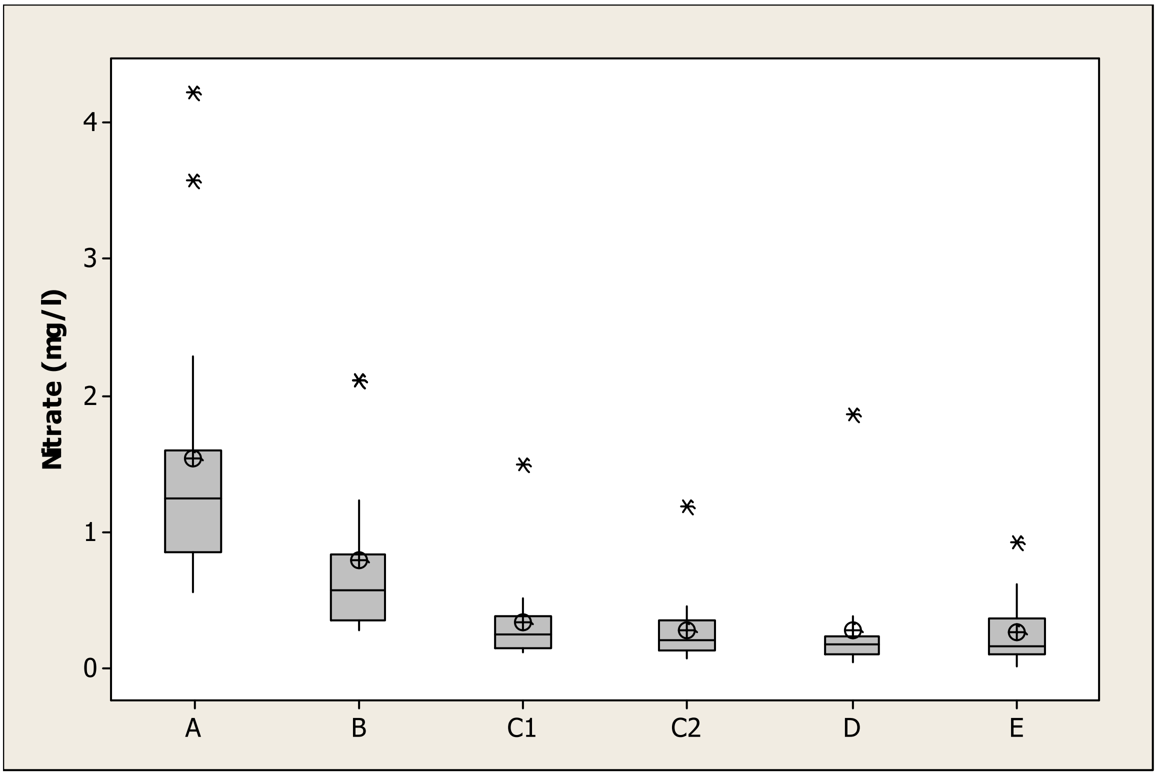

Figure 30: Nitrate values in the treatment steps

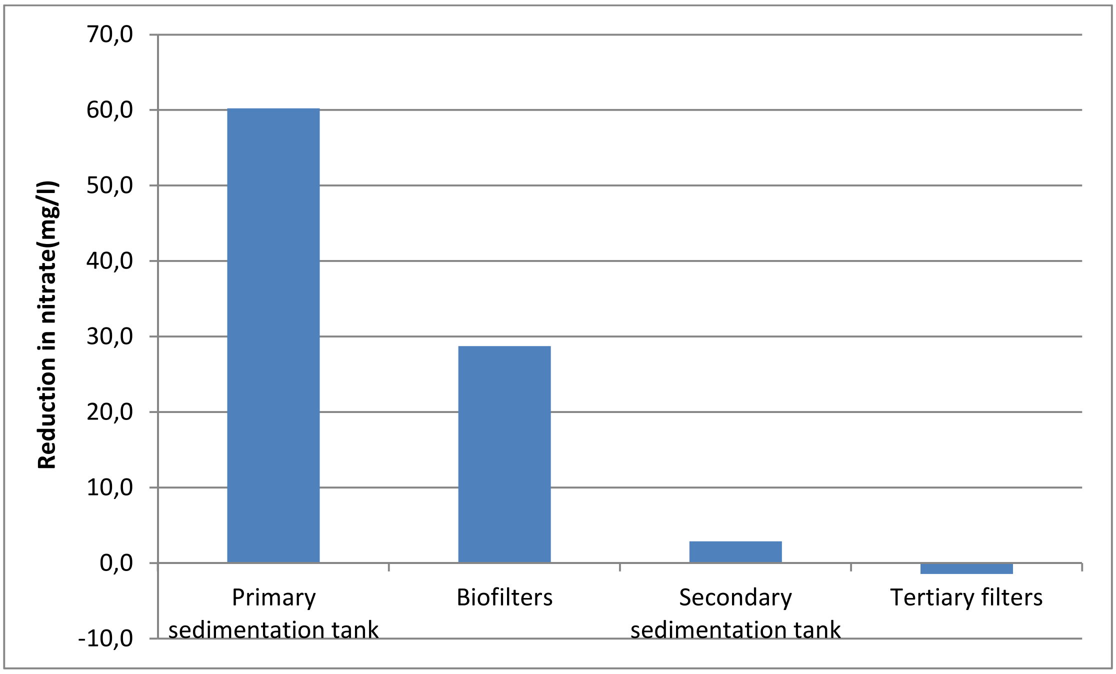

Figure 31: Reduction of initial value of nitrate in each treatment step

4.4 Microbial Parameters

For both E. coli and TFC a total log reduction of 0.55 was obtained in the entire treatment system. The primary sedimentation tank achieved a log reduction of 0.16/0.17 (E.coli/TFC) while the biofilters achieved a log reduction of approximately 0.26/0.25. The observed log reduction of E. coli and TFC in the secondary sedimentary tank was not significant (0.01/0.02). In the tertiary filters a log reduction of approximately 0.12 was found for both E. coli and TFC.

The results of the analyses and statistical interpretation of E. coli and TFC concentrations are presented in figure 32 through figure 34, in addition to table 8 through table 9.

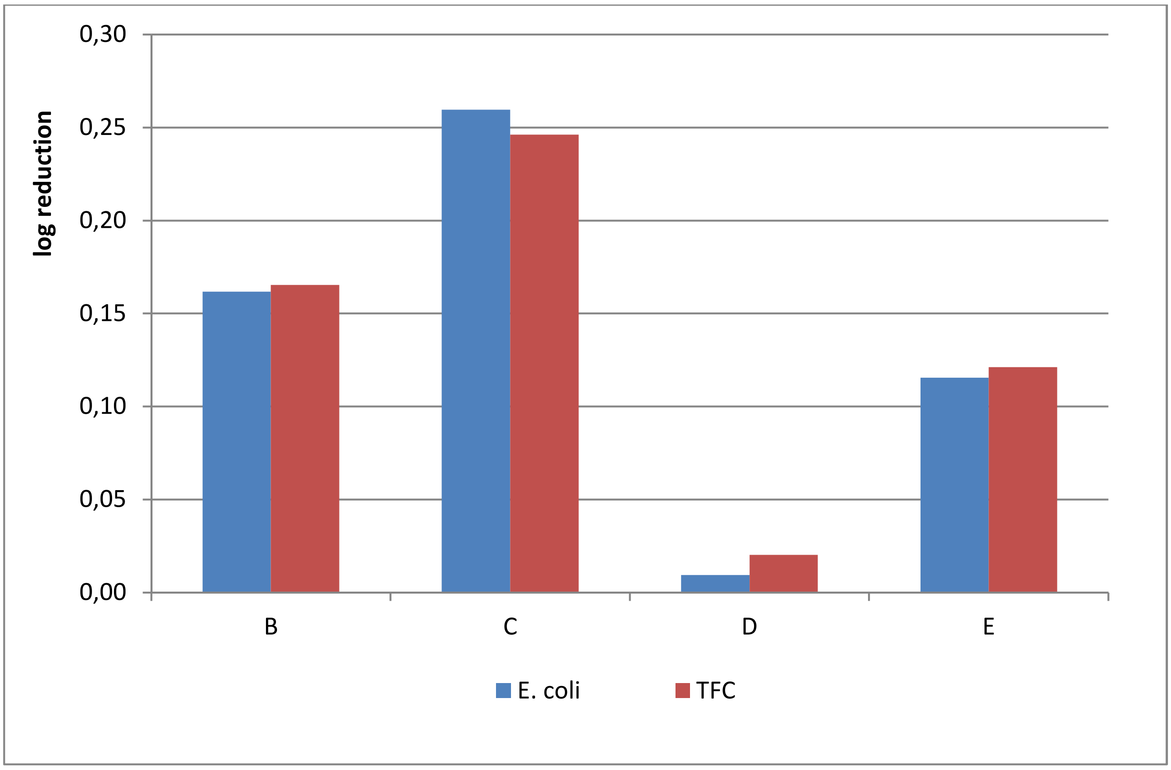

Figure 32: Total log reduction in each treatment step for E. coli and TFC

Table 8: Mean values (MPN/100 ml), standard deviation and range for E. coli analyses from sampling points A-E

SAMPLING POINT	N	MEAN	STANDARD DEVIATION	RANGE	
				MIN.	MAX.
A	7,63E+07	1,05E+08	1,00E+06	3,55E+08	7,63E+07
B	5,25E+07	6,91E+07	5,20E+06	2,48E+08	5,25E+07
C1	3,40E+07	2,63E+07	8,60E+06	7,89E+07	3,40E+07
C2	2,38E+07	1,85E+07	1,00E+06	6,13E+07	2,38E+07
D	2,83E+07	2,34E+07	5,20E+06	9,06E+07	2,83E+07
E	2,17E+07	2,39E+07	1,01E+06	8,13E+07	2,17E+07

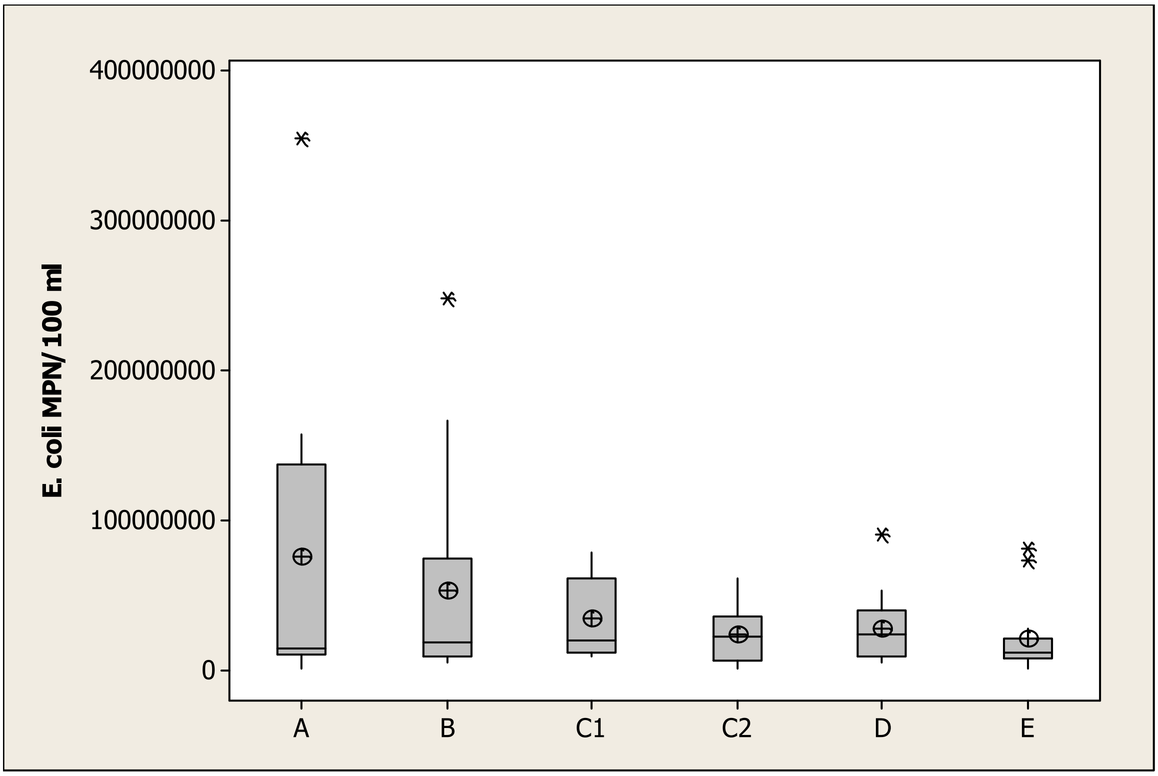

Figure 33: E. coli values observed at the different treatment units of the plant

Table 9: Mean values (MPN/100 ml), standard deviation and range for TFC analyses from sampling points A-E

SAMPLING POINT	N	MEAN	STANDARD DEVIATION	RANGE	
				MIN.	MAX.
A	13	1,40E+08	1,90E+08	1,00E+06	5,91E+08
B	15	9,59E+07	1,05E+08	9,80E+06	3,65E+08
C1	15	6,49E+07	5,76E+07	1,34E+07	2,05E+08
C2	14	4,38E+07	3,50E+07	2,00E+06	1,24E+08
D	15	5,19E+07	3,80E+07	8,60E+06	1,25E+08
E	15	3,93E+07	4,26E+07	1,01E+06	1,54E+08

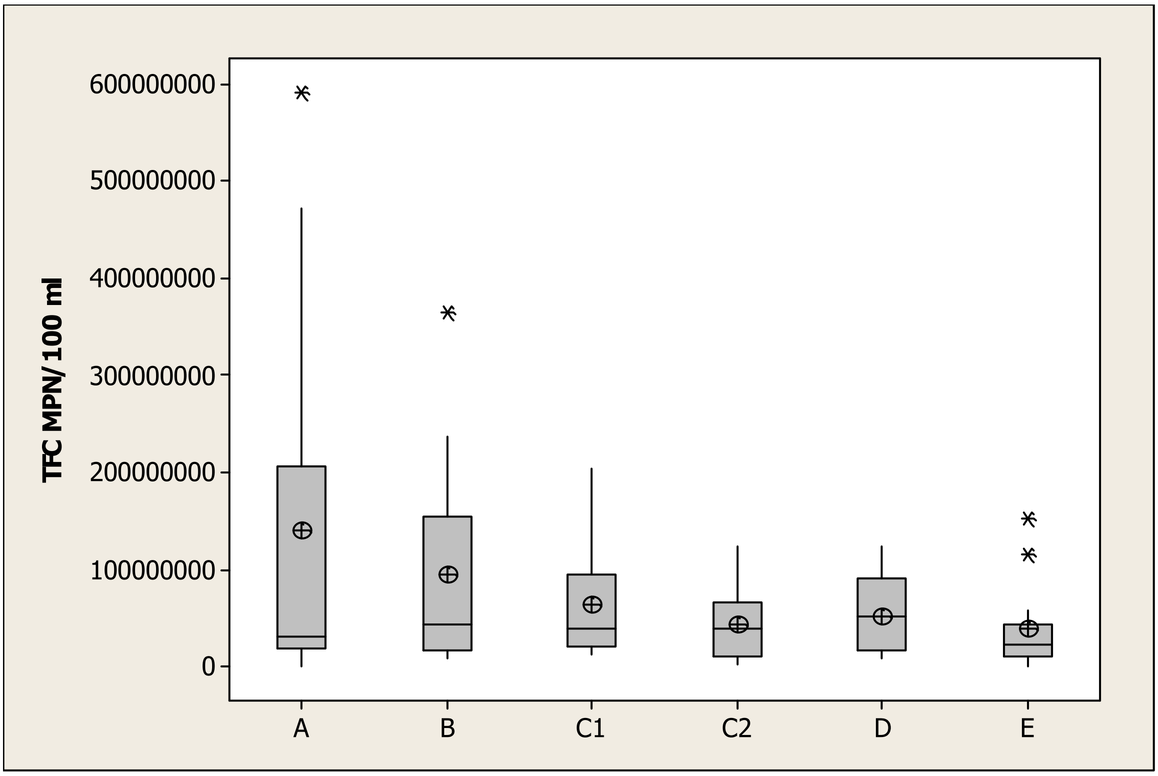

Figure 34: TFC values observed at the different treatment units of the plant

4.5 Regression analysis

A statistical regression analysis was performed to evaluate if any of the physical/chemical parameters affected the concentration of E. coli in the outlet and the inlet. E. coli was used as dependent variable and temperature, DO, pH, BOD, ammonia and nitrate as independent variables.

For this analysis, a p-value < 0.05 indicates a strong statistical relation of the findings. As seen in table 10 and 11 the analysis showed there was no significant relationship between the parameters and E.coli (p > 0.05).

Table 10: Results of regression analysis for the inlet

R^2 = 62.5 %	Constant	Temp.	DO	pH	BOD	NO_3	NH_3
Coefficient	38.03	-0.3097	-0.978	-2.563	-0.000161	0.6081	-0.9076
P-value	0.474	0.704	0.529	0.496	0.929	0.138	0.161

Table 11: Results of regression analysis for the outlet

R^2 = 37.2 %	Constant	Temp.	DO	pH	BOD	NO_3	NH_3
Coefficient	-0.884	0.0209	0.2931	0.9758	-0.000415	0.6005	0.2
P-value	0.916	0.89	0.66	0.255	0.929	0.553	0.871

5.0 Discussion

Although the plant has recently been rehabilitated to improve its performance, this study questions the investments made. The study has shown that the rehabilitated KNUST WWTP is running sub-optimally in the removal of pollutants harmful to the environment and human health. The concentrations of E. coli, TFC and BOD all exceeded the benchmark concentration levels acceptable to the Environmental Protection Agency (EPA) of Ghana. The benchmark concentration of E. coli by the WHO guidelines for irrigation water was also out of reach. The performance of the plant for E. coli and TFC was particularly at such a low level (0.55 log removal) that the utility value of running the plant at the current cost is questionable. These findings are similar to the findings made in 2009 on the same treatment plant, which also gave outlet values for E. coli, TFC and BOD respectively above the EPA benchmark levels (Fosu 2009). This also correspond with the statement by United Nations Development Programme in 2000, that only 30 % of the sewage treatment facilities in the cities of sub-saharan Africa were operating satisfactory (UNEP 2000). The poor performance recorded by the plant can be attributed to a number of technical design and operational considerations. Worth noting though, is that the concentration of both nitrate and ammonia in the effluent of the plant was below the maximum concentration levels acceptable to the Environmental Protection Agency (EPA) of Ghana. The ammonia concentration in the effluent was 0.26 mg/l, compared to the EPA of Ghana limit of 1.0 mg/l. The nitrate concentration was also lower than the maximum permitted outlet value of 50 mg/l both at the inlet and the outlet.

A major finding of the study was that there was a significant overloading of the plant, as the connected population has risen by approximately 3 500 % since it was built with a design capacity for 700 students. The plant today receives wastewater from a population of 25 000. This overloading has several implications for the performance of the plant, mainly as it reduces the retention time of the wastewater in the primary sedimentation tank, causing a less efficient removal of TSS and BOD. For instance the primary sedimentation accounted for only about 10 % of the initial concentration is removed, compared to an expected value of 25-40 % (Metcalf & Eddy 2004). This can be seen as a result of the plant's overloading, together with the lack of maintenance of the treatment unit. Since the sedimentation basin is not cleaned on a regular basis, it increases the chances of clogging and decreases the hydraulic capacity.

Considering BOD removal, the primary sedimentation tank is not performing as expected, as only about 10 % of the initial concentration is removed, compared to an expected value of 25-40 % (Metcalf & Eddy 2004). This can be seen as a result of the plant's overloading, together with the lack of maintenance of the treatment unit. Since the sedimentation basin is not cleaned on a regular basis, it increases the chances of clogging and decreases the hydraulic capacity.

The biofilters considered as the main treatment unit for BOD removal performs well in this regard, with a total removal of approximately 70 % out of the filters. Given a further reduction in the tertiary filters, a total removal of 82.6% of the initial BOD value is achieved in the plant. However, these performances combined failed to achieve the EPA limit of 50 mg/l in the effluent. Given that a large fraction of the BOD removal is related to TSS removal, it is not known if the achieved BOD removal was as a result of this solely, or a combination of removal with the TSS and BOD removal through somehow effective biological treatment. With better performance of the primary sedimentation tank, it is likely that a total reduction of 90 % could be achieved in the biofilters, as this would lower the oxygen demand in the biofilters, thus creating better conditions for growth of biofilm and BOD removal. This would again be beneficial with regards to the removal of E. coli and TFC, as effective growth of biofilm would lead to an extended degree of settlement of microbial matter. An observation made during the study was that the nozzles of the rotating arms of the biofilters did not spread evenly, as some of them didn't produce flow due to clogging. This highlights the problem of poor maintenance of the plant's components, as the rotating arms have probably not been cleaned on a regular basis. The result of this is that the full capacity of the biofilters is not utilized.

Given the level of DO in the wastewater, it holds a very low value at the inlet and through the primary sedimentation tank before a pronounced rise is experienced in the outlet of the biofilters. It is known that aerobic biological treatment is dependent of the access of oxygen to perform well, and this is provided for through the aid of aeration of the filter media. The rise in DO at the outlet could be a result of the wastewater being exposed to free oxygen after leaving the filter media, streaming through the aerated underdrain.

The tertiary filters are designed to promote nitrification, as the wastewater holds a content of BOD normally below10 mg/l (Metcalf & Eddy 2004). In this study, 103 mg/l of BOD is observed in the inlet of the tertiary filters indicating that the conditions are not ideal. A slight increase in nitrate concentration was found, pointing to the possibility that some nitrification has occurred. The tertiary filters gave a log reduction of E. coli/TFC on 0.12, which is much lower than expected. This can be partly due to the fact that backwashing is not practised as often as desired, and not on a regular basis.

Regarding the removal of E. coli and TFC, the performance of the plant was nowhere near the expected or desired value. According to the WHO's guidelines for the safe use of wastewater, excreta and greywater (WHO 2006) a well-functioning wastewater treatment plant is expected to reduce bacterial concentrations by approximately 0-1 log units during primary treatment, 1-2 log units in secondary treatment and 0-3 log units in tertiary treatment. Also, the EPA Ghana standard for the maximum permissible wastewater effluent discharge levels ranges from 0 MPN/100 ml to 400 MPN/100 ml of total coliforms. Less than one log reduction in the whole plant was observed during the study, giving an effluent concentration of $2.17*10^7$/100 ml and $3.93*10^7$/100 ml for E. coli and TFC respectively. This demonstrates that the KNUST wastewater treatment plant does little to reduce microbial health hazards and risk. A large fraction of the E. coli/TFC would normally be eliminated from the wastewater stream as it attach to the biofilm in the biofilters.

Given the poor performance of the plant, it is causing an economic cost without creating benefits, as the goals of treatment efficiency are not reached. Thus, it is essential that action is taken to change the situation. To make favourable changes on the treatment situation, it is important to consider the actual conditions for the treatment plant with regards to technical, financial and institutional issues.

Given the available funding of the KNUST wastewater treatment plant, it is not likely that large investment and operational costs are being released for more advanced high-cost technological systems. Increasing the general operation and maintenance costs through advanced technologies would also make the plant more vulnerable to sudden economic shortcomings. As the training and maintenance practises among personnel are known to be poor, it is not desirable to increase the maintenance requirements of the plant.

As the study shows, the root of many of the problems experienced in the treatment is due to overloading of the primary sedimentation tank. An upgrading of this treatment unit to increase its design capacity would give a smaller fraction of TSS and BOD in the next treatment step. This would improve conditions for biological treatment. As this treatment unit is just mechanical, it would not generate large investment costs, and it would not change the operation and maintenance routines. The initial drawings of the treatment plant show that a future extension was planned, showing that there is available land for extending this treatment unit. The sedimentation tank should be designed so that a detention time of 1.5-2.5 hours is achieved, based on today's flow situation.

For the trickling filters, one possibility of increasing its performance is through changing the filter media from rock to plastic. The benefit of this would be that both higher hydraulic loadings and higher loading rates of BOD would be allowed, as the filter media has a larger specific surface area. Typical values are 45-60 m^2/m^3 for rock and 90-150 m^2/m^3 for plastic (Metcalf & Eddy 2004). An argument for choosing this upgrade possibility is that it won't require any operation and maintenance practices more advanced than those today, and it will not increase the operational costs. A possible disadvantage of changing to plastic media is that the investment cost would possibly be high, depending on the local availability of good quality plastic filter media.

5.1 Limitations of the study

The total number of samples for most parameters in this study was 15. To increase the validation of the findings, the sampling regime could have been carried out over a longer period. This would have made the data series statistically stronger. Still, the decision to end the sampling at the given time was due to a clear trend in the findings for microbial analyses.

Analyses of TSS, total nitrogen and phosphorus were made, but as the results was not released in time they was not used in the study. Daily sludge samples were also started, but were not taken for more than a few days, as the filter bed isolated for this purpose was filled with new sludge several times during the sampling period. With these additions to the study, a more comprehensive analysis of re-use possibilities could have been achieved.

6.0 Conclusion

The study has shown that the KNUST WWTP is running sub-optimally in the removal of pollutants harmful to the environment and human health. The concentrations of E. coli, TFC and BOD all exceeded the benchmark concentration levels acceptable to the Environmental Protection Agency (EPA) of Ghana. The poor removal of E. coli/TFC (0.55 log) is particularly alarming, as it proves that the plant does little to reduce microbial health risks.

These findings complement the trend of poor functionality of wastewater treatment plants in developing countries. The poor performance was traced to a combination of technical, operational and financial factors. Overloading of the plant beyond its design capacity and poor maintenance practices were identified to be the main causes of the plant's poor performance.

Given the cost of running the plant, it is essential that improvements are made to increase the performance. Possible improvements must at least satisfy the following criteria:

- Increase of the plant's hydraulic capacity
- Low investment and operational costs; and
- No further complication of the operation and maintenance procedures

7.0 References

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Appendix

D.O. (mg/l)	07.feb	09.feb	10.feb	11.feb	13.feb	14.feb	15.feb	16.feb	17.feb	21.feb	22.feb	27.feb	28.feb	29.feb	01.mar
A	0,64	0,4	0,15	0,73	0,16	0,34	0,18	0,39	0,43	0,24	0,2	0,91	0,3	0,6	0,11
B	0,51	0,09	0,11	0,83	0,61	0,51	0,13	0,26	0,37	0,21	0,2	1	0,2	0,14	0,12
C1	7,32	6,2	5,79	5,24	4,07	4,3	1,3	5,2	3,88	7,11	5,25	5,7	4,11	7,14	4,11
C2	6,95	6,6	6,4	4,86	3,39	3,22	2,59	5,77	4,35	6,74	4,95	5,62	4,9	7,32	4,35
D	5,25	3,7	1	3,9	0,9	0,12	0,79	1,36	0,81	0,36	1,5	3,4	2,6	1,51	1,57
E	0,93	0,28	0,8	0,27	0,54	1,18	0,12	0,45	0,65	0,12	0,36	0,52	0,33	0,16	0,11
Temp. (˚C)	**07.feb**	**09.feb**	**10.feb**	**11.feb**	**13.feb**	**14.feb**	**15.feb**	**16.feb**	**17.feb**	**21.feb**	**22.feb**	**27.feb**	**28.feb**	**29.feb**	**01.mar**
A	28,5	27,8	26,5	28,1	27,3	27,8	27,7	28,2	27,2	28	27	29,8	27,5	27	27,8
B	28,2	26,4	27,4	27,1	26,9	27,5	27,2	27,8	27,5	27,4	27,1	28,3	27,2	26,9	27,3
C1	23,8	25,7	24,4	27	28,1	27,9	26,7	28,9	28,7	27	27,3	29,1	26,6	26,4	27
C2	25,2	25,3	25,7	26,9	28	28,1	27,9	29,1	29	27,4	27,3	28,9	27,2	26,8	28,2
D	25,2	25,6	25,1	27	28	28,3	28,1	29	28,9	26,6	27	28,9	26,9	26,4	28,2
E	26,3	25,8	26,5	27,4	28	28	28	27,1	28,5	27,1	27,1	30,4	27,3	27	27,6
F	27,9	28	28,1	28,2	27,5	28,1									
pH	**07.feb**	**09.feb**	**10.feb**	**11.feb**	**13.feb**	**14.feb**	**15.feb**	**16.feb**	**17.feb**	**21.feb**	**22.feb**	**27.feb**	**28.feb**	**29.feb**	**01.mar**
A	8,3	8,24	8,67	8,27	8,44	8,44	8,56	8,5	8,51	8,53	8,55	7,85	8,33	8,38	8,3
B	6,42	8,45	7,68	8	7,22	8,22	8	8,27	7,98	7,38	7,6	7,67	7,63	6,72	8,02
C1	7,35	7,96	7,46	7,73	7,88	7,93	7,83	7,76	7,56	7,9	7,79	7,4	7,62	7,54	7,67
C2	7,3	7,93	7,83	7,69	7,86	7,88	7,77	7,75	7,56	7,99	7,83	7,46	7,7	7,38	7,7
D	7,2	7,94	7,77	7,68	7,7	7,85	7,66	7,7	7,52	7,82	7,64	7,41	7,46	7,27	7,63
E	7,17	7,56	7,75	7,63	7,66	7,84	7,91	7,71	7,55	7,9	7,66	7,44	7,28	7,4	7,52

BOD_5 / 100 ml	07.feb	09.feb	10.feb	11.feb	13.feb	14.feb	15.feb	16.feb	17.feb	21.feb	22.feb	27.feb	28.feb	29.feb	01.mar
A	282	312	670		665	440	450	440	330	340	220	490	360	370	480
B	480	300	354	590	570	380	370	270	310	396	204	350	354	390	360
C1	162	141	99	63	120	144	192	78	60	186	54	156	114	126	106
C2	156	153	78	57	147	138	144	66	48	180	48	132	108	126	114
D	210	132	60	69	78	126	72	54	48	168	48	162	60	120	138
E	150	144	45	54	66	102	42	60	42	72	36	90	30	60	96
F	630	336				750									
NO_3/100ml	07.feb	09.feb	10.feb	11.feb	13.feb	14.feb	15.feb	16.feb	17.feb	21.feb	22.feb	27.feb	28.feb	29.feb	01.mar
A	1,276	1,32	2,29		0,88	1,36	4,22	3,58	1,23	0,58	0,78	1,2	1,34	0,88	0,56
B	2,113	0,44	0,57	0,35	2,11	1,23	0,84	0,79	0,46	0,28	0,74	0,34	0,8	0,35	0,42
C1	1,496	0,32	0,39	0,12	0,46	0,51	0,26	0,2	0,25	0,24	0,2	0,13	0,33	0,13	0,15
C2	1,188	0,35	0,21	0,07	0,44	0,46	0,14	0,16	0,24	0,23	0,19	0,12	0,25	0,12	0,15
D	1,87	0,31	0,24	0,04	0,2	0,39	0,06	0,07	0,19	0,18	0,18	0,12	0,2	0,11	0,13
E	0,924	0,35	0,38	0,02	0,62	0,36		0,06	0,2	0,15	0,14	0,11	0,18	0,09	0,11
F															
NH_3/100ml	07.feb	09.feb	10.feb	11.feb	13.feb	14.feb	15.feb	16.feb	17.feb	21.feb	22.feb	27.feb	28.feb	29.feb	01.mar
A	0,42	0,432	1,044		0,88	0,492	1,8	1,056	1,32	0,71	1,58	1,2	1,58	1,56	1,392
B	0,66	0,156	0,408	0,97	0,576	0,48	0,31	0,42	0,3	0,43	0,3	0,32	0,451	0,432	0,38
C1	0,204	0,108	0,048	0,59	0,108	0,108	0,081	0,108	0,06	0,08	0,06	0,13	0,12	0,072	0,08
C2	0,156	0,132	0,024	0,55	0,12	0,108	0,08	0,07	0,06	0,06	0,05	0,12	0,11	0,006	0,06
D	0,096	0,024	0,006	0,48	0,096	0,108	0,06	0,04	0,02	0,04	0,05	0,12	0,11	0,056	0,05
E	0,312	0,108	0,108	0,46	0,019	0,084		0,048	0,02	0,01	0,03	0,11	0,09	0,04	0,05

E.Coli MPN / 100 ml	07.feb	09.feb	10.feb	11.feb	13.feb	14.feb	15.feb	16.feb	17.feb	21.feb	22.feb	27.feb	28.feb	29.feb	01.mar
A	7,50E+07	1,58E+08	1,00E+07	1,46E+08	1,09E+08		3,55E+08		1,00E+07		1,00E+06	8,60E+06	1,34E+07	1,46E+07	1,45E+07
B	1,66E+08	5,20E+07	7,50E+07	1,00E+07	2,48E+08	7,76E+07	5,21E+07	1,85E+07	1,20E+07	8,60E+06	5,20E+06	6,30E+06	1,87E+07	8,40E+06	2,92E+07
C1	3,78E+07	5,83E+07	2,01E+07	6,13E+07	6,70E+07	7,89E+07	7,49E+07	1,10E+07	8,60E+06	2,46E+07	1,10E+07	1,22E+07	1,48E+07	1,32E+07	1,61E+07
C2	3,28E+07		4,73E+07	2,41E+07	4,20E+07	6,13E+07	3,41E+07	1,00E+06	6,30E+06	5,20E+06	1,97E+07	1,22E+07	9,50E+06	5,20E+06	3,27E+07
D	9,14E+06	3,28E+07	5,12E+07	3,23E+07	4,04E+07	9,06E+07	2,41E+07	5,20E+06	6,30E+06	1,83E+07	5,20E+06	2,56E+07	2,03E+07	9,80E+06	5,30E+07
E	1,01E+06	9,14E+06	2,18E+07	2,16E+07	2,01E+07	7,33E+07	8,13E+07	1,19E+07	3,10E+06	7,40E+06	1,08E+07	9,80E+06	2,09E+07	5,20E+06	2,79E+07

TFC MPN / 100ml	07.feb	09.feb	10.feb	11.feb	13.feb	14.feb	15.feb	16.feb	17.feb	21.feb	22.feb	27.feb	28.feb	29.feb	01.mar
A	1,22E+08	1,83E+08	9,70E+07	4,73E+08	2,31E+08	3,10E+07	5,91E+08		1,00E+07		1,00E+06	2,03E+07	1,73E+07	2,62E+07	2,11E+07
B	2,38E+08	1,71E+08	1,35E+08	2,00E+07	3,65E+08	1,55E+08	1,55E+08	3,45E+07	2,43E+07	9,80E+06	1,34E+07	1,22E+07	4,32E+07	1,71E+07	4,35E+07
C1	7,22E+07	7,33E+07	3,09E+07	1,44E+08	9,59E+07	1,40E+08	2,05E+08	1,89E+07	1,34E+07	5,12E+07	2,65E+07	1,75E+07	2,46E+07	2,13E+07	3,99E+07
C2	6,29E+07		6,50E+07	4,88E+07	7,12E+07	1,24E+08	8,13E+07	2,00E+06	9,70E+06	1,21E+07	2,98E+07	2,92E+07	1,69E+07	6,30E+06	5,48E+07
D	1,01E+07	5,75E+07	9,87E+07	7,49E+07	9,60E+07	1,25E+08	6,02E+07	1,34E+07	8,60E+06	2,56E+07	1,87E+07	5,20E+07	3,13E+07	1,61E+07	9,05E+07
E	1,01E+06	1,01E+07	3,44E+07	3,69E+07	5,86E+07	1,16E+08	1,54E+08	1,83E+07	6,30E+06	1,46E+07	2,35E+07	1,89E+07	4,28E+07	1,10E+07	4,26E+07