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CIV 4200: CIVIL ENGINEERING PROJECT II

ASSESSMENT OF THE PERFORMANCE OF THE LUBIGI SEWAGE AND FAECAL SLUDGE TREATMENT PLANT

BY

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Submitted in Partial Fulfillment of the Requirements for the Award of a Degree of Bachelor of Science in Civil Engineering

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DECLARATION

I declare that the work presented in this report is original and has never been presented for award to any academic institution of higher learning. We confirm that where consultations were made either from publications or works of others, it has been cited in the report.

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DEDICATION

I dedicate this report to my loving family and friends for their unbridled support and financial assistance during the course of this research project. I also dedicate it to my lecturers at the School of Engineering, Makerere University who have enabled me to reach this milestone of education.

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LIST OF ACRONYMS

BOD	Biochemical Oxygen Demand
FC	Faecal Coliforms
FS	Faecal Sludge
KCCA	Kampala Capital City Authority
NWSC	National Water and Sewerage Corporation
SDGs	Sustainable Development Goals
TN	Total Nitrogen
TP	Total Phosphorous
TSS	Total Suspended Solids
WSP	Waste Stabilization Pond
WW	Wastewater
WWTP	Wastewater Treatment Plant

ABSTRACT

80% of wastewater released into the environment is without adequate treatment. As such, nearly 2 billion people in the world are exposed to diseases such as dysentery and cholera. One method for treating faecal sludge is by co-treating the faecal sludge with wastewater with a recommended faecal sludge flow rate of 3.6%. The design of the Lubigi treatment plant does not meet this requirement with a FS flow rate of 8% and this could lead to operational problems of the treatment plant and failure to meet effluent requirements.

Samples were picked over a period of five weeks from the influent wastewater and the effluent from the faecal sludge thickening tanks and analyzed for total suspended solids, biochemical oxygen demand, total nitrogen, total phosphorous and faecal coliforms and flow measurements also made to characterize the two wastewater streams. The pond removal efficiencies were also determined by analyzing samples picked from the influent to and effluent from the anaerobic and facultative ponds. The final effluent was also compared to the NEMA discharge standards. The pond geometry and sludge depths were measured.

The influent wastewater quality parameters ranged as follows: TSS; 270-391mg/l, BOD_5; 209.1-622.2mg/l, TN; 70.2 -281.4mg/l, TP; 15.4-84.5mg/l and FC; 4.64×10^5- 4.9×10^6cfu/100ml. The effluent from the faecal sludge thickening tanks quality parameters ranged as follows; TSS; 201.5-321.6mg/l, BOD_5; 1002.9 - 1621.5mg/l, TN; 130.2–311.4mg/l, TP; 77.8–121.5mg/l, FC; 5.1×10^5-1.372×10^6cfu/100ml. The removal efficiencies of the anaerobic ponds ranged from 70-80% whereas the removal efficiencies of the facultative ponds ranged from 82-93% for the different parameters. TSS concentrations met the effluent discharge standards unlike the other parameters. The facultative ponds L:W ratios were slightly above recommendation and the HRTs for the ponds were lower than the design values.

There is accumulated sludge in the anaerobic ponds and the facultative ponds likely have accumulated sludge near the inlet. Effluent from the treatment system is discharged into the environment with high nutrient, BOD_5 and FC contents. Ways of improving the treatment system include increasing the breadth of the facultative pond by about 7 metres, providing multiple inlets to the facultative pond, increasing the desludging frequencies and reducing the faecal sludge flow at the plant.

CHAPTER 1-INTRODUCTION

1. 1 Background

Good water quality is essential to human health, economic and social development and the ecosystem. However, it is likely that over 80% of wastewater (WW) globally is released to the environment without adequate treatment (UNESCO, 2017). This is a likely reason behind the 1.8 billion people that are exposed to diseases such as cholera, typhoid and dysentery because of using a water source contaminated with faeces (WHO/UNICEF, 2015). This shows a slow progress towards achieving the sustainable development goals set by the United Nations in the 2030 agenda (UNESCO, 2017).

In tropical countries, where sewage treatment systems are in use, waste stabilization ponds (WSPs) are usually the chosen option for treatment of WW. WSPs can be used for the co-treatment of WW with the effluent following solid-liquid separation of faecal sludge (FS) in settling-thickening tanks (Bassan, et al., 2014). Experience with the co-treatment of FS with WW in WSPs shows that numerous problems may arise as a result (EAWAG/SANDEC, 1999). These problems may include excessive biochemical oxygen demand (BOD) loading rates which lead to odour problems at the facultative ponds, ponds may fill up quickly and the development of algae in facultative ponds may be impaired (EAWAG/SANDEC, 1999).

Faecal sludge (FS) is a term used to refer to the solids and liquids which are removed from a pit, tank or vault in a wet sanitation system (Tayler, 2018). FS is about 50 times as concentrated as domestic sewage in terms of organic and solids loading (USEPA, 1984). The quantity of FS that a plant can handle is governed by the nature of flow of the FS. The flow of FS relative to the sewage is important since it determines the additional organic solids load on the treatment plant. Appropriate facilities are needed at the treatment plant to receive, pretreat and distribute the FS into the specified process units. The performance of a sewage treatment plant accepting FS is dependent on many factors which include the type of process units, design capacity and volume of FS added daily among others (USEPA, 1984).

According to Strande et al. (2014), co-treatment of FS with wastewater is not recommended for the vast majority of cases in low-income countries. However, a FS flow of 3.6% of the maximum plant design capacity is allowed. Since many wastewater treatment plants (WWTPs) in developing countries may either operate below design capacity, as the supporting sewers are not yet in place, or far above capacity (e.g. at 130% level) as infrastructure upgrading did not keep pace with the increase of sewer connections, regulations have to consider the facility performance limits. These limits depend on the manner and part of the process in which FS is introduced (Jayathilaka et al., 2019).

1. 2 Problem Statement

The National Water and Sewerage Corporation (NWSC) treatment plant in Lubigi has a design capacity of 400m^3 and 5000m^3 for faecal sludge and wastewater respectively per day (KCCA, 2014). It co-treats wastewater from a network of sewers with effluent following solid-liquid separation of FS in settling-thickening tanks from areas of Mulago, Bwaise, Kawempe, and Makerere among others. The majority of wastewater treatment plants in low-income countries such as Ghana have failed and improper co-treatment with FS has even been reported as the cause of some failures since WWTPs are typically not properly designed for received FS loadings (Bassan, *et al*., 2014). Often, plant operational problems and deteriorated removal efficiencies arise due to high BOD, TS and NH_4 concentrations typical of faecal sludge. In addition the pathogenic quality of effluent (helminth eggs, faecal coliforms) is also impaired (EAWAG/SANDEC, 1999). Therefore, various biological treatment processes are affected by faecal sludge overloading. Lubigi Sewage treatment plant was designed to co-treat wastewater and faecal sludge through a series of waste stabilization ponds (WSPs). However, the design has a FS flow rate of 8% which exceeds the recommended capacity of 3.6% for co-treatment. Furthermore, the FS treated at Lubigi treatment plant is of high strength (includes waste from public toilets) yet the 3.6% flow rate was recommended for septage which is much lighter waste. In addition to that, the 400m^3 /day faecal sludge flow was meant to be reduced to 300 m^3 /day from the year 2020 after completion of the Nalukolongo FS treatment plant (NWSC,2009). Since the Nalukolongo treatment plant has not been completed, the Lubigi treatment plant receives more FS flow (660 m^3/day) than it was designed to handle (KCCA, 2020). This shows that the recommended FS flow is far exceeded necessitating the checking of the efficiency of the treatment plant. The treatment processes at Lubigi treatment plant should therefore be studied to ensure they are efficient and the effluent released to the environment meets the effluent discharge standards (NEMA, 1999).

1. 3 Main Objective

The main objective of this study was to assess the performance of the Lubigi sewage and faecal sludge Treatment plant.

The specific objectives of the study were,

i. To characterize the wastewater influent and effluent from the FS thickening tanks.
ii. To determine the treatment efficiency of the Waste Stabilization Ponds (WSPs).
iii. To determine ways of optimizing the WSP treatment processes in relation to the treatment of FS thickening tank effluent with WW.

1.3 Significance

This study aimed at determining the efficiency of the Lubigi treatment plant in treating the influent wastewater and effluent from FS thickening tanks. The results of this study will inform the design and operation of other treatment plants that co-treat wastewater with effluent from FS thickening tanks and improve the water situation of 80% of wastewater released to the environment without adequate treatment, bringing countries closer to attaining a large number of SDGs.

1.4 Study Scope

The study focused on assessing the performance of Lubigi treatment plant in treating faecal sludge effluent from the thickening tanks with wastewater. The quality characterization of the wastewater was limited to total nitrogen, total phosphorous, biochemical oxygen demand (BOD_5), total suspended solids (TSS), faecal coliforms and chlorophyll a parameters. The quantity characterization involved measuring flows. The treatment plant is located in Namungoona in Kampala, Uganda. The plant co-treats two streams, the first being WW and the second, FS. The sewage stream comprises of screening, grit removal, anaerobic and facultative ponds and unplanted drying beds for sludge. The effluent from the facultative ponds discharges into the Lubigi wetland. The FS stream comprises of screening and grit removal chambers, covered thickening tanks, covered drying beds and covered storage areas for sludge. The liquid effluent from the thickening tanks is co-treated with the WW in the WSPs. Figure 1.1 below shows a map along with an aerial view of the plant.

Figure 0.1: An aerial view of Lubigi Sewage Treatment Plant

CHAPTER 2-LITERATURE REVIEW

2.1 Introduction

The co-treatment of faecal sludge (FS) effluent with wastewater (WW) can reduce the treatment efficiency of plants. This can result in increase in the organic loading caused by the faecal sludge. This chapter focuses on the current literature written on wastewater and faecal sludge treatment necessary for the study. It includes pretreatment, treatment in waste stabilization ponds (WSPs), pond geometry, operation and maintenance of treatment processes, and various parameters for measurement.

2.2 Wastewater Pretreatment

Wastewater is the used water from a combination of domestic, industrial, agricultural or commercial activities, surface runoff or storm water and sewer infiltration if any. This is transported through the sewers to the treatment plant before discharge into the environment (Tayler, 2018). Preliminary treatment ensures a satisfactory quality of final effluent and final sludge product and protects the treatment process from malfunction due to accumulation of screenings, debris, inorganic grit, excessive scum formation or loss of efficiency associated with grease or oil films or fat accumulations (EPA, 1995). The processes of pretreatment include screening, removal of grit, oil, grease and fat.

2.3 Faecal sludge Pretreatment

2.3.1 Screening and grit removal

Faecal sludge (like WW) goes through separate screening and grit removal units. According to Bassan, *et al.,* (2014) bar screens are placed where the influent comes in from. The bar screens at the influent remove municipal solid waste and large solids both the faecal sludge and wastewater that helps in prevention of clogging and pump failures. Bar screens are either placed vertically or inclined against the incoming flow hence making a physical barrier that retains the coarse solids and lets the liquid to go through. After going through both the coarse and fine screens, then they go through the grit chamber that helps in the removal of grit. According to Tayler (2018), faecal sludge contains a high concentration of grit. This high content in grit content increases the rate at which sludge accumulates in the ponds and may also damage the mechanical equipment.

2.3.2 Settling-thickening Tank

Faecal sludge is then taken into the sedimentation tank that separates the solids from the liquid flow. To ensure proper flow in the sedimentation tanks, scraper mechanism is used to push sludge that settles along the length of the tank back to the sump. (Tayler, 2018). At Lubigi treatment plant, solidified faecal sludge is sent to thickening chamber and then to the drying beds. The liquid effluent is then connected to the influent wastewater into the anaerobic pond. The efficiency of settling-thickening tanks with respect to removal of total suspended solids (TSS) can reach up to 80% where the tanks have been adequately designed and operated (Bassan, *et al*., 2014).

2.4 Wastewater and faecal sludge treatment in ponds

Wastewater stabilization ponds (WSPs) are large, shallow rectangular basins where there is a continuous inflow and outflow of the domestic wastewater, septage and sludge as well as animal and industrial waste. WSPs are frequently used in combination with other sanitation technologies. The most common types of WSPs are anaerobic ponds, facultative ponds and maturation ponds. The treatment processes in the ponds are highly influenced by the favorable climatic conditions which is one of the key factors in the treatment process since the WSPs operate usually under purely natural biological processes. These are usually low-cost (least cost), low maintenance, high efficient in removal of organic pollutants and highly sustainable hence usually used in most of the low developing countries (Picot, *et al*., 2005).

2.4.1 Co-treatment of wastewater and faecal sludge

Wastewater can be co treated with the liquid effluent of the faecal sludge from settling-thickening tanks in WSPs as done at Lubigi treatment facility. However, care should be taken during the addition of the effluent from the faecal sludge thickening tanks not to exceed an amount (3.6%) as this could lead to a decline in the efficiency of the WSPs due to increased organic load and ammonia concentration (Bassan, *et al.*, 2014). The efficiency of anaerobic and facultative ponds in the treatment of the organic pollutants of this mixture (wastewater influent and faecal sludge liquid effluent) was determined at Lubigi treatment facility. Characterization of influent raw wastewater quality is critical for the selection and design of an appropriate treatment technology. It is also necessary for evaluating the performance of separate unit processes and operations (Dawood *et al*., 2017).

2.4.2 Anaerobic ponds

The primary function of anaerobic ponds is stabilization and breakdown of the high concentrations of organic pollutants contained in wastewater and not necessarily production of a high-quality of effluents (Farzadkia, *et al*., 2014). These are deep shallow ponds that exclude oxygen hence encouraging the growth of bacteria that helps in the breakdown of effluent. These ponds are used as a pretreatment for BOD, TSS, and COD removal. They also remove pathogens to a small percentage (Tayler, 2018). Anaerobic ponds are commonly 2 – 5 m deep and receive wastewater with high organic loads (i.e., usually greater than 100 g BOD/m^3day, for a depth of 3 m). In anaerobic ponds, BOD removal is achieved by sedimentation of solids, and subsequent anaerobic digestion in the resulting sludge. The process of anaerobic digestion is more intense at temperatures above 15ºC. The anaerobic bacteria are usually sensitive to pH <6.2. Thus, acidic wastewater must be neutralized prior to its treatment in anaerobic ponds. A properly-designed anaerobic pond will achieve about a 40% removal of BOD at 10º C, and more than 60% at 20º C. A shorter retention time of 1.0 - 1.5 days is commonly used (Kayombo, *et al*., 2004).

2.4.3 Facultative ponds

Facultative ponds are 1-2 m deep with a long detention time (2-3 weeks) that makes them more efficient in bacteria removal. There are two types of facultative ponds: Primary facultative ponds that receive raw wastewater, and secondary facultative ponds that receive particle-free wastewater (usually from anaerobic ponds, septic tanks, primary facultative ponds, and shallow sewerage systems). The process of oxidation of organic matter by aerobic bacteria is usually dominant in primary facultative ponds or secondary facultative ponds (Bassan, *et al*., 2014). The processes in anaerobic and secondary facultative ponds occur simultaneously in primary facultative ponds. Their main purpose is to remove organic material and solids but they can also remove ammonia that is incorporated into biomass by use of biological process. Facultative ponds are designed for BOD removal on the basis of a relatively low surface loading (100 – 400 kg BOD/ha.day) (Kone & Peter, 2010). It is estimated that about 30% of the influent BOD leaves the primary facultative pond in the form of methane. A high proportion of the BOD that does not leave the pond as methane ends up in algae. This process requires more time, more land area, and possibly 2 -3 weeks hydraulic retention time, rather than 2-3 days in the anaerobic pond. In the secondary facultative pond (and the upper layers of primary facultative ponds), sewage BOD is converted into algal BOD which, has implications on effluent quality requirements. About 70 – 90% of the BOD of the final effluent from a series of well-designed WSPs is related to the algae they contain. The pH> 9 is obtained in the pond after some biological processes, which can kill faecal coliform hence reducing the pathogens levels in the pond (Kayombo, *et al*., 2004)

2.4.4 Maturation Ponds

Maturation ponds are usually the third of the WSPs in the treatment process. They receive effluent from facultative ponds and their size and number depends on the required bacteriological quality of the final effluent (Quiroga, 2005). They are typically shallower than facultative ponds with a depth in the range of 1-1.5 metres. These ponds are also well oxygenated throughout their depths because they receive lower organic loads than the anaerobic and facultative ponds. Algal populations are also much more diverse in maturation ponds than in facultative ponds and the diversity increases from pond to pond in the series (Varon *et al.*, 2004). Maturation ponds are usually designed majorly to remove excreted pathogens (WHO, 1987). They also make a significant contribution to nitrogen and phosphorous removal and to a smaller extent achieve removal of BOD (Varon *et al.*, 2004).

2.4.5 Pond geometry

The pond geometry influences the sludge sedimentation patterns with them, turning the water movement and mass dispersion. There has been little rigorous work done on determining optimal pond shapes and they vary considerably in their geometry. The most common shape is rectangular, although there is much variation in the length-to-breadth ratio and it should be 2:1 (Tayler, 2018). In general, anaerobic and primary facultative ponds should be rectangular, with length-to-breadth ratios of less than 3, so as to avoid sludge banks forming near the inlet (Tayler, 2018). Secondary facultative and maturation ponds should, wherever possible, have higher length-to-breadth ratios (up to 10, or even 20 to 1) so that they better approximate plug flow conditions. High length-to-breadth ratios may also be achieved by placing baffles in the pond.

A single inlet and outlet are usually sufficient, and these should be located in diagonally opposite corners of the pond. The use of complicated multi-inlet and multi-outlet designs is unnecessary and not recommended. To facilitate wind-induced mixing, the pond should be located so that its longest dimension (diagonal) lies in the direction of the prevailing wind. If this is seasonally variable, the summer wind direction should be used as this is when thermal stratification is potentially maximal. To minimize hydraulic short-circuiting, the inlet should be located such that the wastewater flows in the pond against the wind. (WHO, 1987). Despite the possibility of reading the dimensions of the WSPs from the design report, this study included the actual measurement of pond dimensions. This was done due to operation and maintenance (O & M) activities that could cause the ponds to operate at depths different from the design depths for example sludge accumulation reduces the design depth in anaerobic and facultative ponds thereby diminishing the effective pond volume, potentially causing overturning and the resuspension of settled pathogens, reducing overall pathogen removal (Verbyla, *et al*., 2017). Figure 2.1 shows a schematic of the WSPs with recommended dimensions.

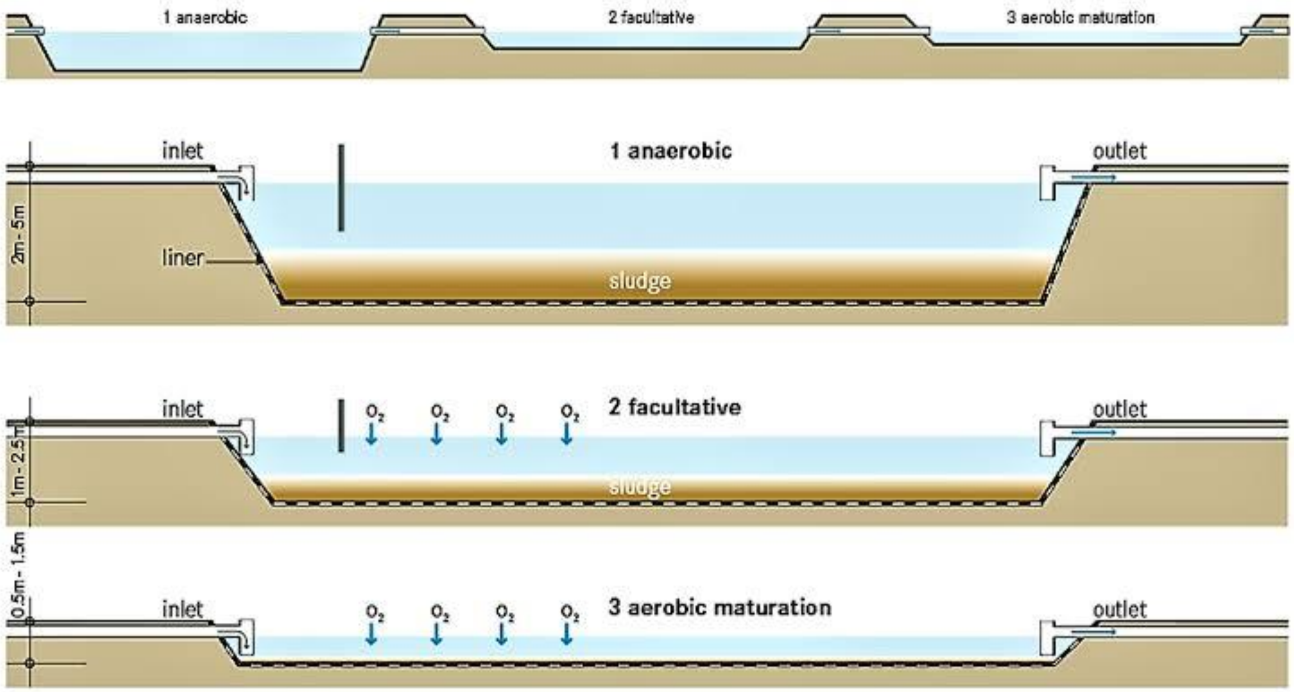


Figure 0.1: Schematic of waste stabilization ponds

The depth chosen for any particular pond depends on site considerations (presence of shallow rock, minimization of earthworks). In primary facultative ponds, especially those with high length-to-breadth ratios, it is often advantageous to provide a deeper zone (2-5 m) near the inlet for sludge settlement and digestion.

2.4.6 Operation and maintenance of WSPs

The main operational measures that the WSPs require include; the withdrawal of sludge and the control of odors through the recirculation process of pond effluent from final ponds, according to Quiroga, (2005) who determined that the ponds need to be desludged every 2 to 3 years to ensure their proper performance. When the system is already running and the construction of the pond is already free of vegetation it is important to know that the waste stabilization pond is not waterproof, and should be filled with raw wastewater and seeded with bio-solids from another anaerobic reactor. Gradually the anaerobic ponds can be loaded periodically from one to four weeks, depending on the quality of the digester used. It is important to remember that in the first month it is necessary to add lime to avoid acidification of the reactor (Quiroga, 2013). Once the WSPs start operating, it is necessary to carry out the maintenance work. In maintaining the ponds to serve their purpose, the optimum loading rate of the ponds is determined hence ensuring that sludge does not accumulate to form a thick sludge layer that would require desludging of ponds before they are fully drained (Bassan *et al*., 2014). According to Kengne *et al.*, (2011) the loading rate of 100 kg TS/m^2/year, results in the accumulation of about 30-40 cm/year of sludge, compared to 50-70 cm/year if the loading rate of 200 kg TS/m^2/year is used. For ponds with a freeboard of 1.5 m to 2 m these loading rates would result in a 3-5 year operation life before desludging is required. At this time when desludging is required, the costs of operation and maintenance combined can be exceeded in that year.

2.4.7 Desludging of ponds

Desludging is a process through which sediments in the ponds are removed through draining and cleaning of the ponds. This is essential whereby if not carried out when the ponds are past a third full of their capacity, need might arise for periodically taking them out of service for desludging in order to perform effectively and prevent the risk of odours. Both anaerobic and facultative ponds need periodic desludging though in facultative ponds it's required less frequently. This process will be carried out by use of hands or a tractor more often since the sludge in these ponds will often be too thick for pumping (Tayler, 2018). Since desludging is required more frequently in the anaerobic pond, it is therefore advisable to have two parallel anaerobic ponds so as to allow one pond to be desludged.

2.5 Wastewater and faecal sludge parameters

2.5.1 Solids

The solids in the mixture of wastewater effluent and liquid effluent can be either organic (volatile) or inorganic (fixed) and can be either suspended (those that are not able to pass through a filter) or dissolved (those that pass through the filter). The suspended solids include floating material, settleable material and colloidal material while the dissolved solids are in solution (Rost, 2018). The size of the solid particles depends on the source of the sludge and the prior treatment. Solids content of the wastewater effluent and liquid effluent will vary, depending on local conditions such as ambient temperatures that are favorable for the bacteria (Doulaye & Strauss, 2004). Treatment mechanisms involve the removal of suspended solids by the sedimentation process. The suspended solids from facultative ponds are approximately 60−90 per cent algae. The algal content that is present in the ponds contributes to relatively high BOD and TSS levels in the effluent compared with other treatment processes. Facultative ponds treating wastewater have reported TSS removal efficiencies of 70–80% (Doulaye & Strauss, 2004).

2.5.2 Biochemical oxygen demand

BOD_5 is a measure of the oxygen demand exerted by the readily bio-oxidizable organic material during their decomposition by bacteria and other microorganisms contained in a wastewater sample over a given time period. It is used in the assessment of the quality of the water. This parameter is obtained from the amount of oxygen, divided by the volume of the system, taken up through the respiratory activity of microorganisms growing on the organic compounds present in the sample i.e. water or sludge, when incubated at a specified temperature (usually 20^0C) for a fixed period (usually 5 days, BOD_5) (Jouannue, 2014). Liquid effluent from the faecal sludge has a higher BOD_5 value than that of strong wastewater. Wastewater is considered to have BOD_5 values that range between 200 and 700mgl (Heinss *et al.*, 1999). For such loading of the anaerobic pond at the treatment plant, the performance and efficiency of the pond was assessed. Non carbonaceous material can also consume oxygen, for example during nitrification, which can increase the reported BOD_5 value if not taken into account. The particle size distribution also has an effect, as smaller and more soluble particles have faster BOD_5 reaction rate coefficients. Other factors that can account for sample variability include sample filtration, dilutions, and sampling methodologies (Bassan, et al., 2014).

Treatment is needed to reduce the wastewater's extremely high oxygen demand and suspended solids concentration to levels that do not effect environmental contamination of the receiving water (Tayler, 2018). These ponds work extremely well in warm climates, with the removal of BOD_5 ranging from 60-85% in a very short retention time in the anaerobic pond. The WSPs are normally placed ahead of a treatment line involving secondary facultative and maturation ponds. (Quiroga, 2005). Anaerobic ponds reduce microorganisms by sludge formation and the release of ammonia into the air. This treatment also serves to breakdown biodegradable organic material. BOD_5 and COD removal efficiencies drop more rapidly with time than solids removal efficiencies although the decrease in the COD clarification effect is less significant than that of BOD_5 (Heinss *et al.*, 1999). The COD/BOD_5 ratio decreases through the transformation of non-biodegradable COD to biodegradable COD (with the concurrent increase in BOD_5) When treating municipal wastewater, correctly sized, configured, and operated facultative ponds can remove 70–90 per cent of the influent BOD_5 that wasn't removed in the anaerobic pond (Doulaye & Strauss, 2004).

2.5.3 Nutrients

Most of the nutrients found in the household faecal and wastewater sludge include, nitrogen and phosphorous. The faecal and WW sludge contain up to 0.7% nitrogen a percentage of wet weight which is about 5 to 11 g per day. Incase these nutrients are released to the environment in an uncontrolled manner, they will cause eutrophication and contamination of the environment (Rost, 2018). Ammonia can also be toxic to a variety of fish in relatively low concentration. The concentration of nitrogen leaving the preliminary treatment is an important factor in determining the size and the cost of the entire system (Sherwood, 1984). Nitrogen exists in wastewater in different forms which include primarily organic nitrogen, ammonia and nitrate. Nitrogen concentration in typical municipal wastewater ranges from 15 to greater than 50 mg/l. Under favourable conditions, WSPs can achieve up to 80% removal of nitrogen (Sherwood, 1984). Organic nitrogen is hydrolyzed to ammonia in anaerobic ponds after which the ammonia is incorporated into algal biomass in facultative and maturation ponds (Kayombo, *et.al.*, 2004). Typical influent wastewater contains a total phosphorous concentration of 5-9 mg/l. Phosphorus exists in various types in wastewater such as orthophosphate, polyphosphate and organically bound phosphates. Total Phosphorus includes soluble and particulate phosphorus. Phosphorous is removed through uptake by algal biomass, precipitation and sedimentation. According to Kayombo *et.al.* (2004), the best way to remove much of the phosphorous is to increase the number of maturation ponds. This implies that for efficient removal of phosphorous, more land area is required.

2.5.4 Pathogenic microorganisms

Pathogens are always present in both untreated and partially treated wastewater and liquid effluent from faecal sludge. The release of untreated or partially treated WW into the environment has negative effects if there is any reuse of the effluent produced. Faecal sludge contains pathogenic microorganisms namely; bacteria, viruses, protozoa and helminths (Bassan *et al.*, 2014). The concentration of helminth eggs in the biosolids is largely dependent on the prevalence and intensity of infection in the population from which FS or wastewater is collected (Kone & Peter, 2010). The pathogens occur in raw faecal, final effluent and water environments (Dias *et al*., 2018).. When humans get in contact with for example polluted water or food, these pathogenic organisms can cause illnesses, which is a concern worldwide. Diarrhoea, hepatitis and fever are some of the consequences that can affect humans (Bassan *et al.*, 2014). Such waterborne diseases become a problem when using wastewater for irrigation since this wastewater can lead to the spreading of pathogenic microorganisms. As for pit latrines that retain faecal material for several years, both its volume and the concentration of pathogens decrease during this time. A distinction is often made between high-strength faecal sludge and lower strength septage, with the strength defined in terms of oxygen demand and suspended solids concentration. This distinction is qualitative, rather than quantitative, and should not obscure the fact that both faecal sludge and septage exert a high oxygen demand, have high solids content, and contain large numbers of pathogens (Tayler, 2018).

CHAPTER 3- MATERIALS AND METHODS

3. 1 Introduction

This chapter consists of the different methods that were used to address the objectives of this study. It includes the methods for sampling, laboratory analysis, pond geometry measurements, site conditions and data analysis.

3.2 Flow measurement

Flow of the wastewater at the sampling points was determined by measuring the depth of water upstream of the weirs and then using the formula;

$$\frac{2}{3} \times Cd \times B \times \sqrt{2g} \times H^{\frac{3}{2}} \quad \text{...} \text{Equation 1}$$

Where Cd is the coefficient of discharge
B is breadth of the weir
H is the depth of the wastewater

The coefficient of discharge values for the various sampling points were determined using the Rehbok formula Cd= 0.611+0.08(h/p) where h is the head over the weir crest and p is the weir height. This led to a coefficient of discharge of 0.6 for the pond outlets and effluent from the faecal sludge thickening tanks. A value of 0.48 was used as the coefficient of discharge for the samples taken at the grit removal chamber as guided by the design report.

3.3 Sampling strategy

Samples of wastewater were collected from the grit removal chamber, effluent from FS thickening tanks, influent to anaerobic ponds, outlets from the anaerobic and facultative ponds as shown in Figure 3.4. Composite samples were collected to obtain a representative result because concentration of the analytes may vary over short time periods. The composite samples were obtained by combining portions of five grab samples of 500ml each collected at 2 hour intervals over an 8-hour sampling period i.e. 8:30am, 10:30am, 12:30pm, 2:30pm and 4:30pm (Ontario, 2016). A one litre jerrycan cut at the top was attached to a long pole to collect the samples after which they were placed in sampling containers and covered immediately. Since the inlet of the wastewater is not accessible at the treatment plant, samples were collected from the grit removal chamber (Figure 3.1). Samples of the faecal sludge effluent from the settling-thickening tank were taken from the exit chamber of the thickening tanks. Samples for the influent to the anaerobic pond were taken from the distribution chamber (where the effluent from the FS thickening tanks is combined with the wastewater from the grit removal chamber). Similarly, samples were picked from the outlets of the anaerobic and facultative ponds to be used in determining the efficiency of the treatment units. To take into account the weekly variation in influent and effluent quality of wastewater, samples were collected on five days over a five-week period from 13th February 2020 to 13th March 2020 (WHO, 1987) resulting in a total of five samples per sampling location. The study was carried out in a dry season.The samples were then kept in a cool box (with ice packs at 4°C) and transported to the College of Natural Sciences Laboratory located in Makerere University for analysis.

Figure 0.1: Sample wastewater collection from the grit removal chamber

3.4 Measurement of site environmental conditions

Data on the minimum and maximum ambient temperatures, wind speed, and rainfall measured at Makerere University weather station (because it was nearest to the treatment plant) during the sampling times was obtained from the Uganda National Meteorological Centre situated in Luzira. This data is relevant because the weather conditions have an impact on the treatment process.

3.5 Laboratory Analysis

The collected samples were analyzed for Total Suspended Solids (TSS), BOD_5, chlorophyll-a, nutrient content(Total Nitrogen (TN) and Total Phosphorous (TP)) and faecal coliforms (FC).

TSS was determined by filtering a well-mixed sample through a pre-weighed standard glass-fiber filter and then dried at 105 ^{o}C for at least an hour in a memmert oven (APHA, AWWA & WEF, 2012). BOD_5 was measured using the dilution method, 5-day BOD test which measures the change in dissolved oxygen concentration caused by micro-organisms as they degrade organic matter in a sample incubated for 5 days in the dark at 20^{o}C (APHA, AWWA & WEF, 2012). The persulfate digestion method was used to determine TP and TN according to APHA, AWWA, WEF (2012). After digesting, the readings of TP and TN were read using the colorimeter Hach D/890. Faecal coliforms were determined using the Membrane Lauryl Sulphate Broth (MLSB) according to APHA, AWWA, WEF, (2012). The medium with the WW was autoclaved for 24 hours at 121^{0}C. Chlorophyll-a was determined using the spectrophotometric method at the College of Natural Sciences, Makerere University (APHA, AWWA & WEF, 2012).

3.6 Pond Geometry Measurements

The physical characteristics of the treatment units such as shape, volume, inlet to outlet alignment, depth of each pond were determined through field survey data. Lengths and widths of the ponds were measured to obtain the length to width (L:W) ratios and inlet and outlet depths determined. Sludge depths in the anaerobic and facultative ponds were determined using the 'white towel' test of Malan (WHO, 1987). White toweling material was wrapped on one third of a sufficiently long pole and then lowered vertically into the pond until it reached the pond bottom. It was then slowly withdrawn and the depth of the sludge layer was determined using a measuring tape. By subtracting the sludge depth from the pond depth, the level of sludge in the pond was estimated (Figure 3.3). The sludge depth was measured at five points in each pond away from the embankment base and the mean depth calculated (WHO, 1987). The design report was reviewed to ascertain the design pond dimensions.

Figure 0.2: Measuring sludge depth using the white towel test

3.7 Data analysis

The results obtained from the laboratory were synthesized using Microsoft excel 2010 to develop bar graphs of inlet and outlet wastewater parameters (BOD, TSS, TN, TP, chlorophyll-a and FC) against their concentration to illustrate the removal efficiencies. The removal efficiency of the ponds was determined using the formula;

$$\frac{Influent\ load - effluent\ load}{Influent\ load} \times 100\%$$

Where load = flow × concentration

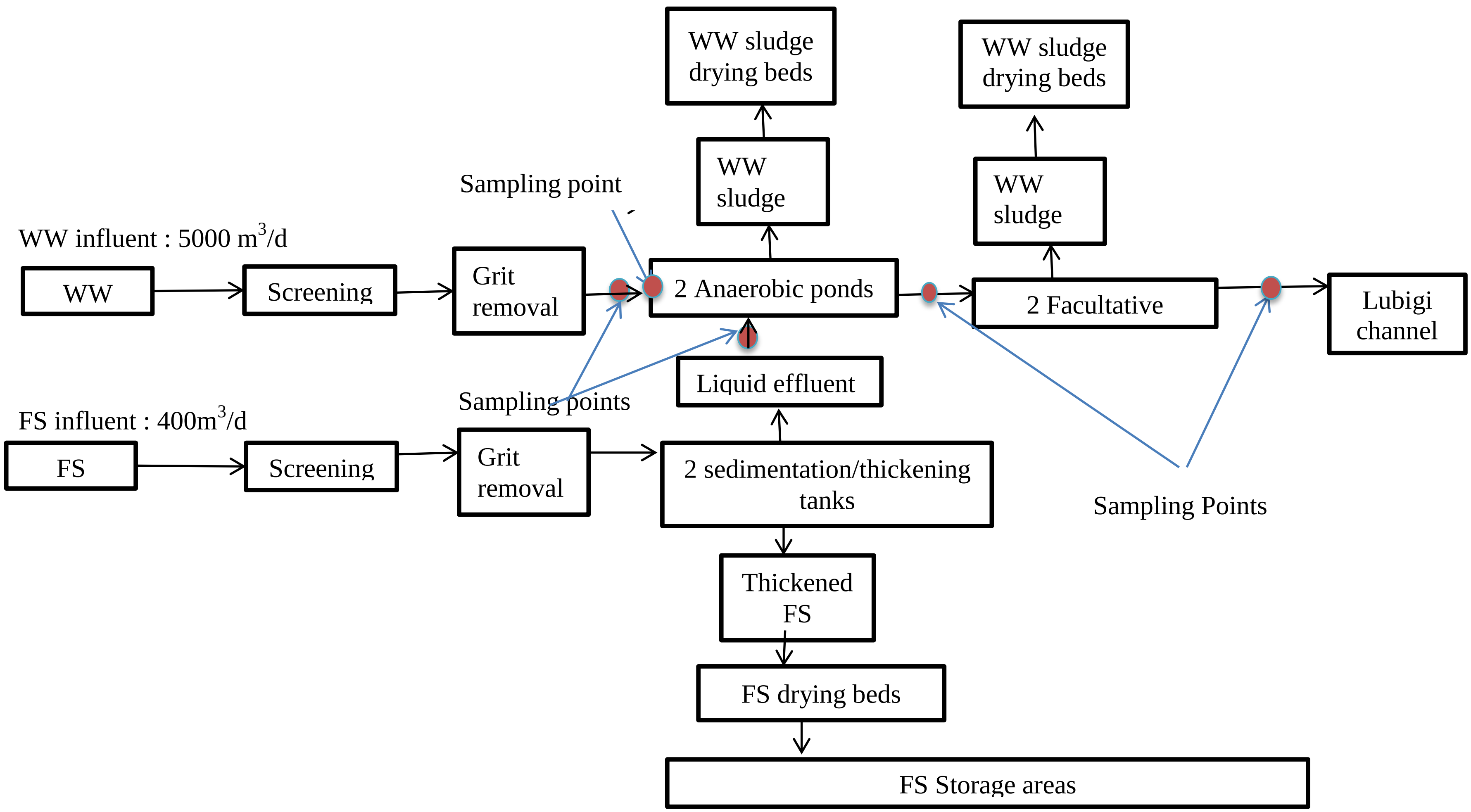


Figure 3. 1: Sampling points at Lubigi Treatment Plant

CHAPTER 4 - RESULTS AND DISCUSSION

4.1 Characterization of Influent Wastewater and Effluent from Faecal Sludge thickening tanks

The section presents the results for flow rate, the quality of influent wastewater (WW) and effluent from faecal sludge (FS) thickening tanks. The section also presents the variation of the two wastewater streams along with the discharge standard.

4.1.1 Flow Rate Results

Results of flows are presented in Table 4.1. The study noted the average flow rate of influent wastewater to be in the range of 2845m^3/day to 4057m^3/day with a median 3433.6 m^3/day and the average flow rate of the effluent from FS settling thickening tanks to be in the range of 138.2 m^3/day to 518.4 m^3/day and a median of 414.7 m^3/day.

Table 4.1: Flow results during sampling period

Date	Influent Wastewater flow (m^3/d)	Effluent from FS thickening tanks (m^3/d)
13/02/2020	3433.6	345.6
19/02/2020	4057	518.4
25/2/2020	2845.8	138.2
2/3/2020	3283	414.7
13/3/2020	3678.7	501.1

4.1.2 Quality of influent raw wastewater and effluent from faecal sludge thickening tanks

The quality of the wastewater is shown in Figures 4.1-4.5 for total suspended solids (TSS), biochemical oxygen demand (BOD_5), total nitrogen (TN), total phosphorous (TP), and faecal coliforms (FC). Details of these results are presented in Table 4.2 (Appendix 1) for the different sampling dates. The BOD_5 ranged from 209 to 623 mg/l with a mean value of 412.6±189.6 mg/l. According to Sperling, (2007), BOD_5 concentration for municipal wastewater ranged from 250-400mg/l with a typical value of 300mg/l. The design report stipulates a design value of 320mg/l. Therefore the BOD_5 concentration measured was slightly higher than the design value. From the measured data, it was found that the total suspended solids (TSS) ranged from 271 to 392 mg/l with a mean value of 336.9±48.7 mg/l. This data was fairly similar to the design value of 320mg/l and to a typical value by Sperling, (2007). The total nitrogen (TN) concentration varied from 70 to 282 mg/l with an average value of 196.1±86.9 mg/l. This was significantly higher than the design value of 80mg/l and a typical value of 45mg/l (Sperling, 2007).The concentration of total phosphorous (TP) in this study ranged from 15 to 85 mg/l with a mean value of 56.9±28.1 mg/l. This is also higher than the design value of 24mg/l. The high concentrations of TP and TN could be due to the WW containing human waste. It was found that the average value of the faecal coliforms (FC) concentration was 2.373×10^6 $\pm1.893\times10^6$ cfu/100ml ranging between 4.64×10^5 and 4.9×10^6 cfu/100ml. The quality of the effluent from the FS thickening tanks is also presented in Figures 4.1-4.5 for similar parameters as mentioned above. The details of these results are presented in for the different sampling dates. The BOD_5 ranged from 1002 to 1622 mg/l with a mean value of 1344.8±312.7 mg/l. This was fairly similar to what was designed for by (NWSC,2009) which was a value of 1760mg/l. The TSS concentration ranged from 201 to 322 mg/l with an average value of 245.5±52.6 mg/l which was considerably lower than the design value of 3040mg/l The concentration of TN in the thickening tanks effluent ranged from 130 to 311 mg/l with a mean value of 248.7±72.9 mg/l. It was found that the average TP concentration was 95.4±21.1 mg/l ranging from 77 to 122 mg/l. TN and TP measurements were below the design values according to the (NWSC,2009). The FC concentration varied from 5.1×10^5 to 1.372×10^6 cfu/100ml with a mean value of $9.362\times10^5 \pm 3.618\times10^5$ cfu/100ml which was higher than the design value of 1.0E+05cfu/100ml.

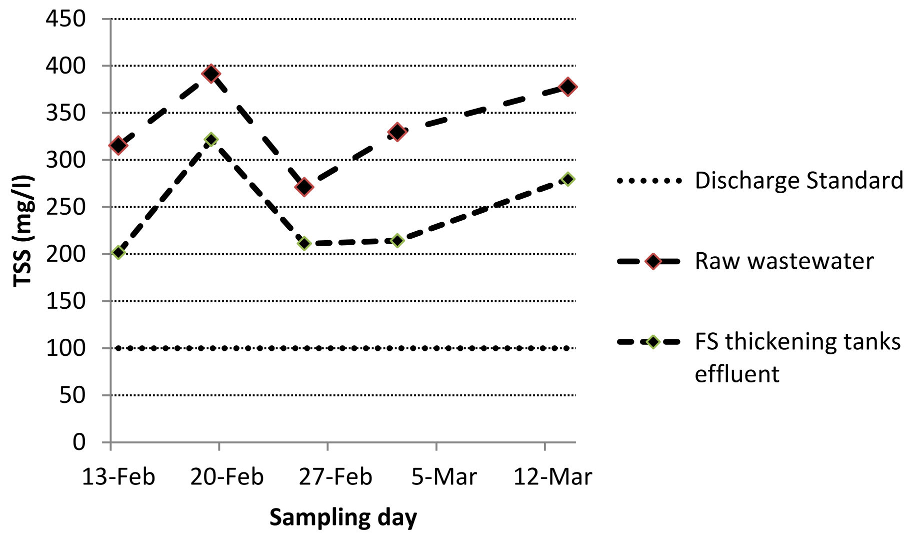


Figure 0.1: Variation of TSS in raw wastewater and faecal sludge thickening tanks effluent with time

From Figure 4.1, the TSS concentration for both the raw wastewater and the FS effluent exhibit a fairly similar trend despite the wastewater exhibiting higher levels than the FS effluent for all the sampling dates which is likely due to the FS thickening tanks where settling of suspended solids occurs. The concentration increased sharply from sampling day1 to day 2, dropped rapidly for day 3 and gradually increased for days 4 and 5. However, the flow rates (for both WW and FS) were considerably low on sampling day 3 as indicated in table 4.1 and could have been the reason for the drop in the concentration on day 3. In the same regard, the highest concentrations were noticed for days with the highest flow rates (days 2 and 5). The TSS concentrations for both the WW and FS were above the NEMA discharge standard of 100mg/l for all the sampling dates meaning there is need for further treatment of the two streams of wastewater before being discharged into the environment.

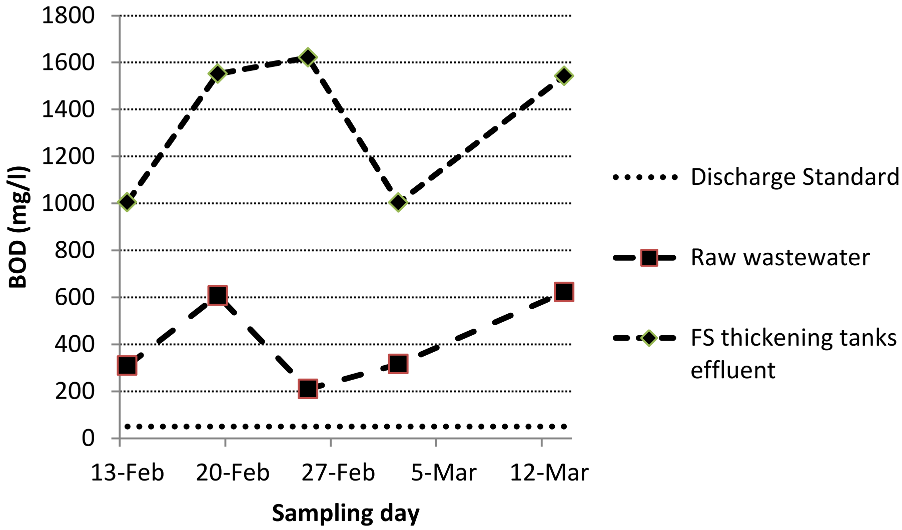


Figure 0.2: Variation of BOD in raw wastewater and faecal sludge thickening tanks effluent with time

From Figure 4.2, for all sampling days, the BOD_5 concentration of the FS thickening tank effluent was significantly higher than that of WW. This is likely so because FS is reported to be much more concentrated than municipal WW i.e. has 10 to greater than 100 times higher contents of organic pollutants (NWSC,2009). The FS thickening tank effluent BOD_5 concentration increased steeply from sampling day 1 to day 2 and slightly increased on day 3. It then dropped sharply on day 4 and increased again on day 5. The trend for the WW was slightly different. It increased from day 1 to day 2, dropped on day 3 and then increased gradually on days 4 and 5. This variation could be attributed to the high flows recorded on 19th Feb and the low flows recorded on 25th Feb. The BOD_5 concentrations for both the WW and FS were above the NEMA discharge standard of 50mg/l for all the sampling dates meaning there is need for further treatment of the WW and effluent from FS thickening tanks

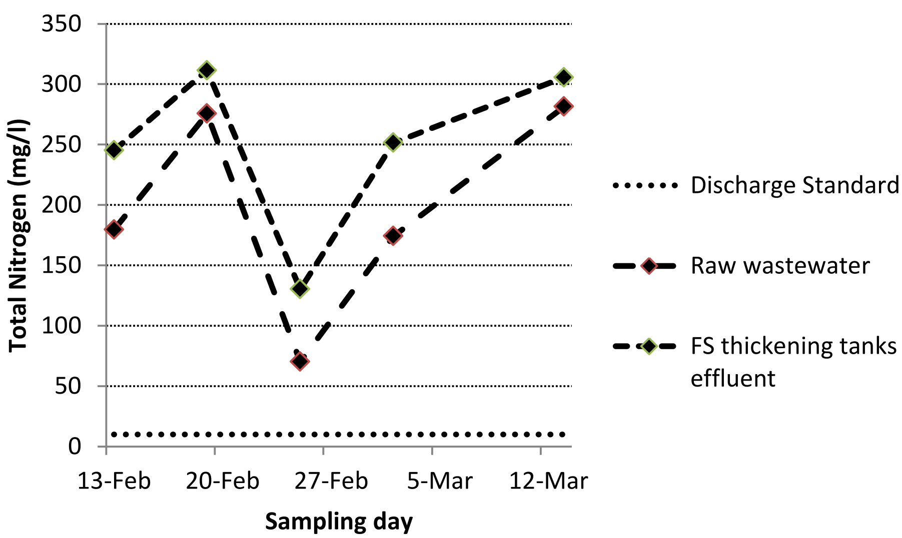


Figure 0.3: Variation of TN concentration of raw wastewater and faecal sludge thickening tanks effluent with time

Figure 4.3 exhibits a similar trend for TN concentration for both the raw wastewater and the FS effluent from the thickening tanks with the latter having higher values throughout the sampling. This is because FS largely contains excreta which comprises of up to 20% of nitrogen in faeces and 90% in urine of the food consumed (Bassan, *et al*., 2014). The concentration increased noticeably from sampling day1 to day 2, dropped dramatically for day 3 and steeply increased for days 4 and 5. The dramatic drop in concentration on day 3 can be owed to the considerably low flow rates observed on day 3 for both the FS effluent and WW and similarly high flow rates observed on days 2 and 5 could explain higher concentrations as shown in Table 4.1. This was expected. The TN concentrations for both the WW and FS were above the NEMA discharge standard of 10mg/l for all the sampling dates meaning there is need for further treatment of the WW and effluent from FS thickening tanks.

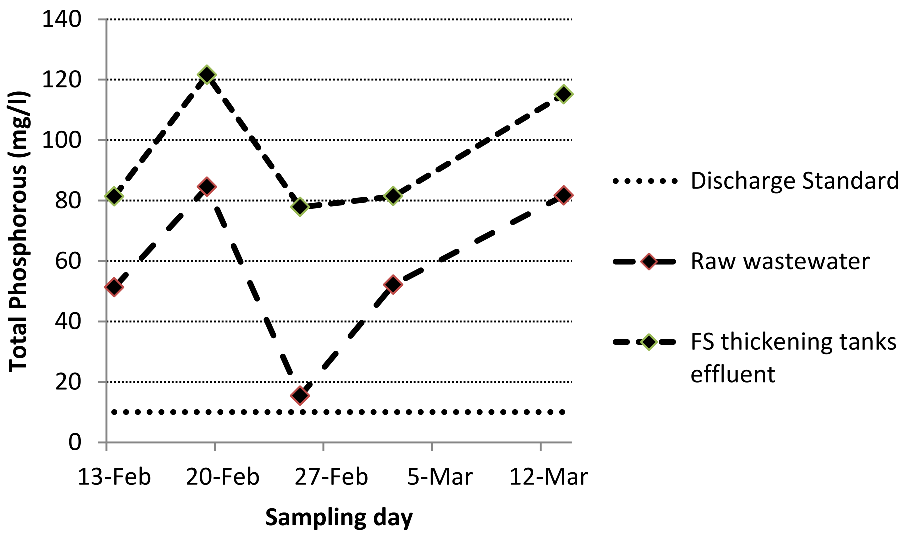


Figure 0.4: Variation of TP in raw wastewater and sludge thickening tanks effluent with time

Figure 4.4 shows a fairly similar trend in the TP concentration for both the raw wastewater and the FS effluent from the thickening tanks with effluent from FS thickening tanks exhibiting higher values than WW. This is because excreta, a major component of FS contains up to 50% of the phosphorous in the food consumed (Bassan, *et al*., 2014) The concentration increased noticeably from sampling day1 to day 2, dropped for day 3 and increased for days 4 and 5. The dramatic drop in concentration on day 3 could be due to the relatively low flow rates observed on day 3 for both the FS effluent and WW as shown in Table 4.1. The TP concentrations for both the WW and FS were above the NEMA discharge standard of 10mg/l for all the sampling dates meaning the effluent from the FS thickening tanks and the raw WW is not yet ready for discharge into the environment at this stage and is in need of further treatment.

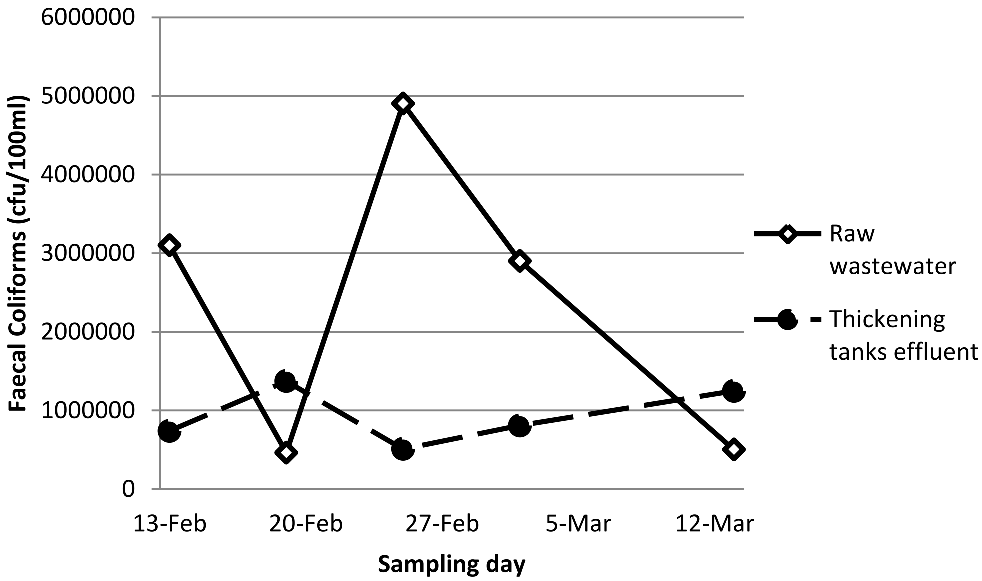


Figure 0.5: Variation of FC of raw wastewater and sludge thickening tanks effluent with time

Figure 4.5 is with a difference in the trends of the FC concentration for WW and FS effluent. Here the WW is largely with higher FC counts for almost all the sampling days than the effluent from the faecal sludge thickening tanks. The FS thickening tank effluent varied closely around 1×10^{6} cfu/100ml throughout all the sampling days while that of WW varied quite widely. This is likely because the raw WW may occasionally contain fresh waste from pit latrines emptied into the drains in the nearby slums of Bwaise which would sharply increase the FC concentration of the WW. The FC concentration for both FS effluent and WW were both high above the NEMA discharge standard of 5000 cfu/100ml for all the sampling dates which implies that the WW and effluent from the FS thickening tanks requires further treatment before it is discharged into the environment.

4.1.3 Environmental conditions results

Table 4.3 shows the state of the environment during the sampling period. It is clear that the sampling was carried out in the dry season. According to Shatat *et.al*, (2014), proper efficiency of waste stabilization ponds (WSPs) is expected at high as opposed to low temperatures for example the BOD removal rate is directly proportional to temperature.

Table 4.3: Environment conditions during sampling period

Sampling day	Rainfall (mm)	Average Temperature (°C)	Maximum Temperature(°C)	Minimum Temperature(°C)
13/02/20	Trace	23.5±2.84	29.2	19.8
19/02/20	0.5	22.7±2.65	28.9	19.1
25/02/20	0.1	22.1±1.66	26	19.5
2/03/20	0.1	22.2±1.44	25.3	20.4
13/03/20	3	22.3±2.03	26.7	19.0

4.2 Pond Geometry

4.2.1 Length to Width Ratio

Anaerobic ponds; 65/30 = 2.2
Facultative pond; 170/50 = 3.4
The length to width ratios were 2.2:1 and 3.4:1 for the anaerobic ponds and facultative ponds respectively. The L:W ratios for the ponds corresponded to the design values. The anaerobic ponds comply with WHO (1987) which recommended a length to breadth ratio of less than 3 for anaerobic and primary facultative ponds. However, the facultative ponds have a ratio of 3.4 which could cause formation of sludge banks near the inlet (WHO, 1987). The side slopes for the anaerobic ponds and facultative ponds are 1:1.5 and 1:2.5 respectively and are made of concrete to prevent erosion by wave action (WHO, 1987).

4.2.2 Inlet and Outlet Positions

The inlets and outlets were located at diagonally opposite corners of the anaerobic and facultative ponds as shown in the figures 4.7 and 4.8 from the design below which complied with the WHO (1987) recommendation in order to minimize short circuiting. The inlets were located below the liquid level in the ponds. Discharging below the liquid level minimizes short circuiting and reduces the quantity of scum in the ponds (WHO, 1987).

Schematics of the anaerobic and facultative ponds are shown below with all dimensions used to calculate the length to width ratios, inlet and outlet positions and side slopes from the design.

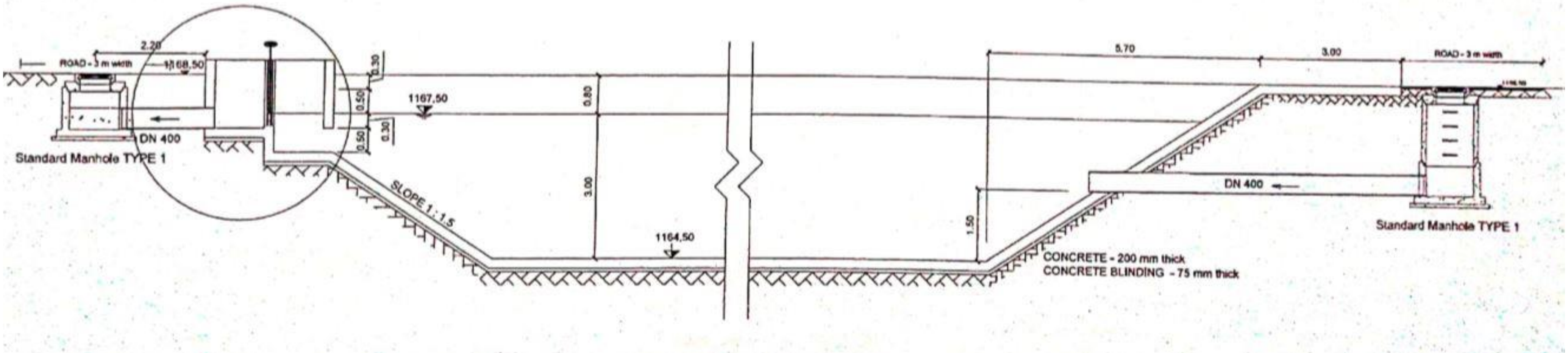


Figure 0.6: Section/vertical profile of the Anaerobic Ponds

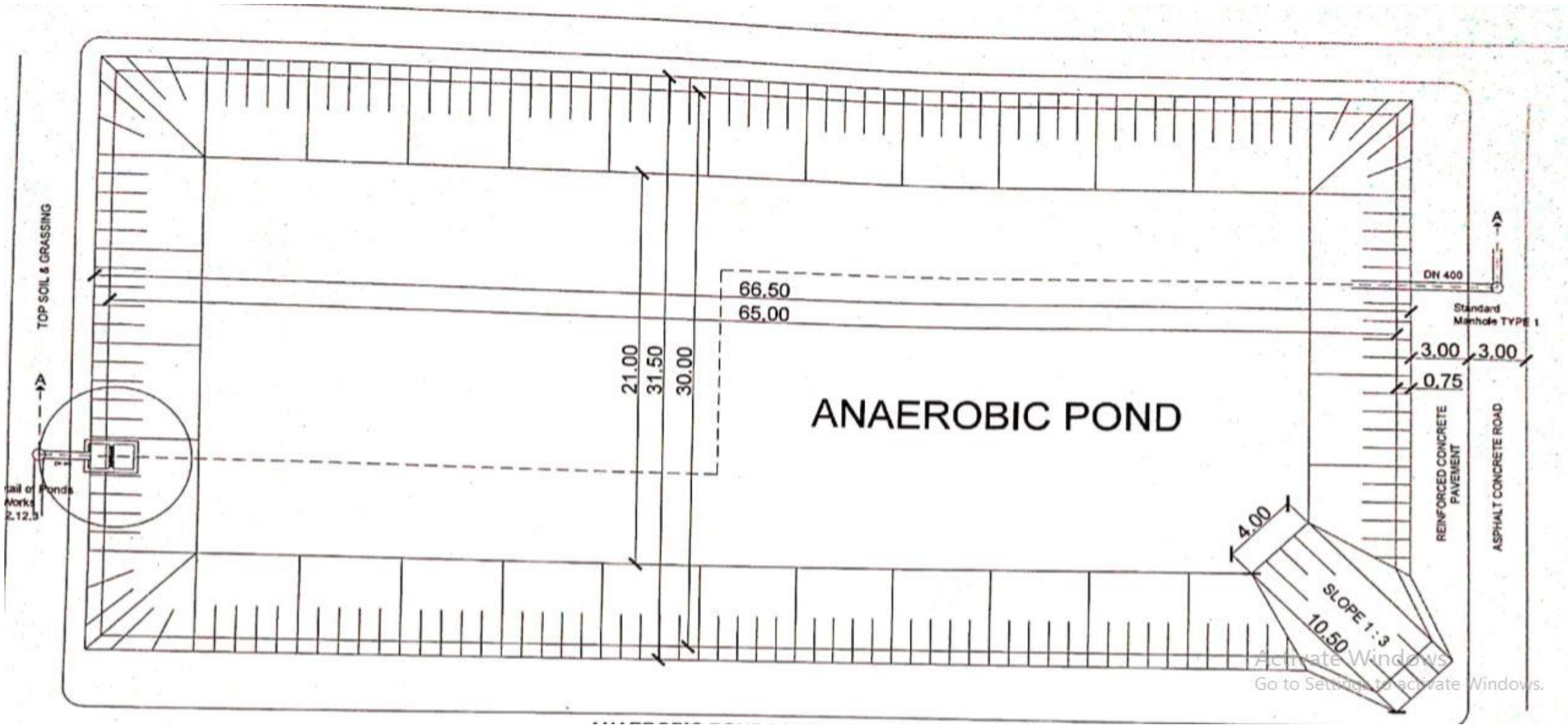


Figure 0.7: Layout plan of the anaerobic ponds

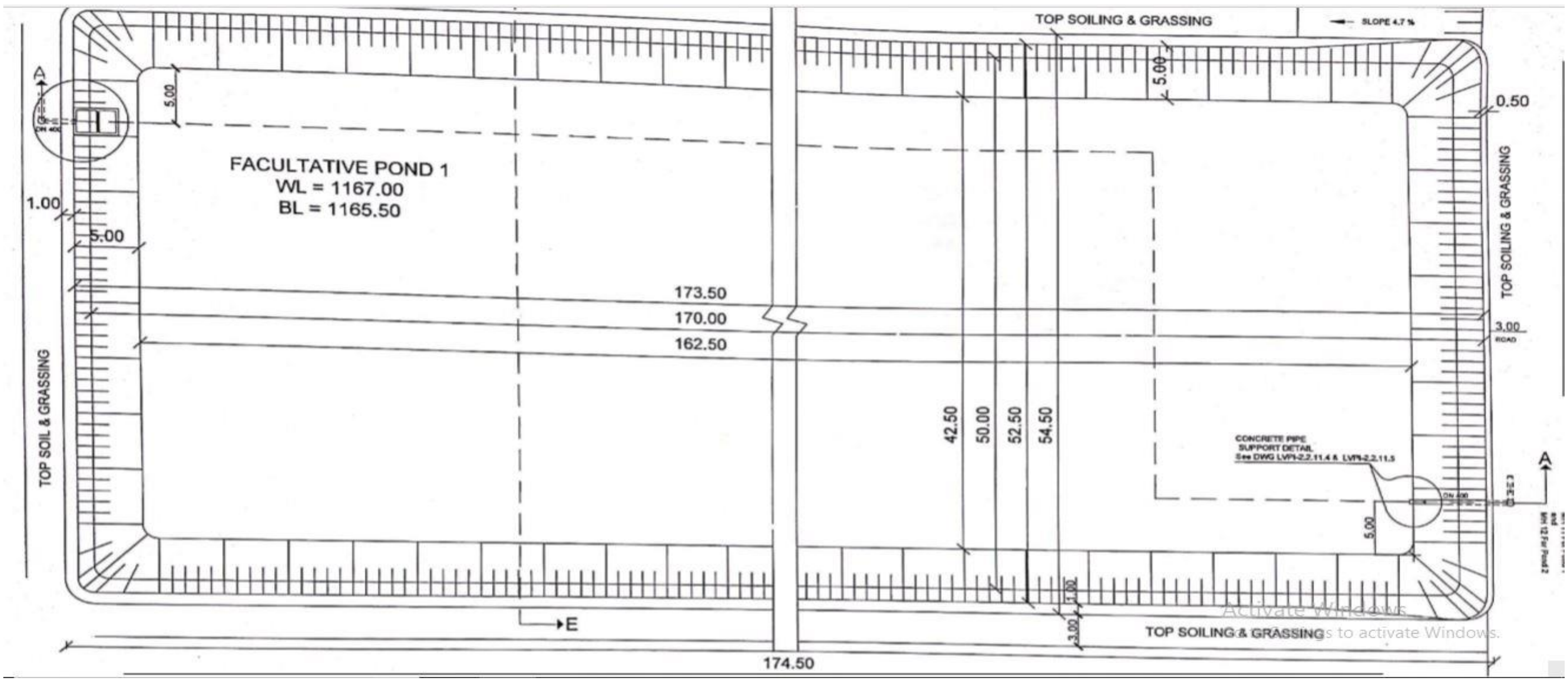


Figure 4.8: Layout plan of facultative ponds

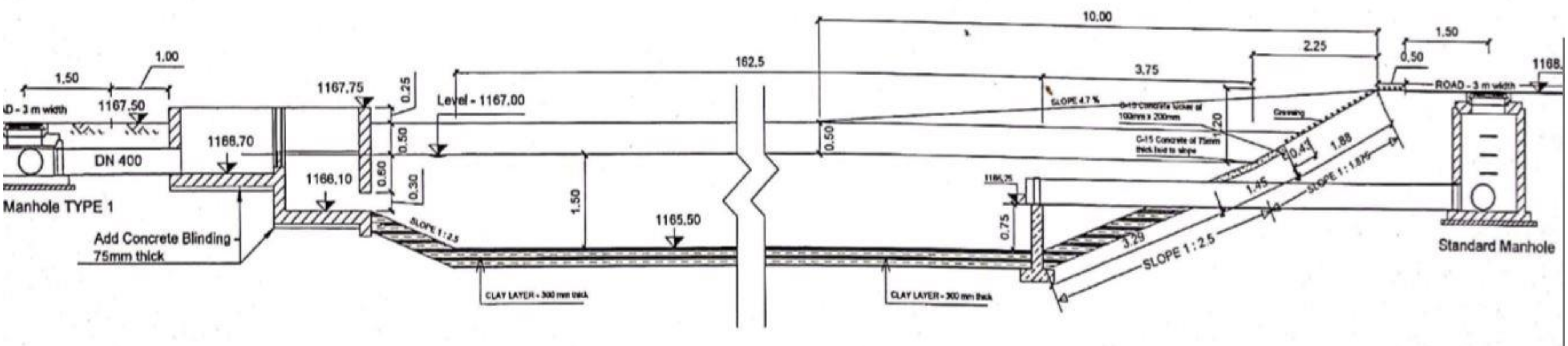


Figure 4.9: Section/vertical profile of facultative ponds

4.2.3 Inlet and Outlet Depths

The recommended outlet depths are 300mm and 600mm for anaerobic and facultative ponds respectively (Spellman & Drinan, 2014). Table 4.4 shows the inlet and outlet depths of the ponds at the treatment plant as designed by NWSC (2009). The depths complied with the recommendation. The outlet in anaerobic ponds should be deep enough to avoid any surface crust but higher than the sludge levels whereas in facultative ponds, the depth is such that discharge is from below the maximum depth of the algal band (Spellman & Drinan, 2014).

Table 4.4: Inlet and Outlet depths for the ponds

	Inlet depths	Outlet depths
Anaerobic Ponds	1100mm	300mm
Facultative Ponds	350mm	600mm

4.3 Sludge Depths/Sludge Blanket Heights

The sludge depths are shown in Table 4.5. The anaerobic ponds were found to have higher sludge blanket heights as compared to the facultative ponds. This is the case because the anaerobic ponds receive influent with more solids which settle immediately and are biodegraded thus the formation of sludge. Therefore there is a reduction in the total volume of the ponds. In this study, volume reductions were 33% and 28.7% for anaerobic and facultative ponds respectively. Due to sludge accumulation, it is possible that the effective hydraulic retention time (HRT) and the overall performance of the system are decreased (Gopolang & Letshwenyo, 2018).

Table 4.5: Sludge measurement heights in the ponds

Ponds	Anaerobic	Facultative
Design depth (m)	3	1.5
Actual depth (m)	2.01	1.07
Sludge blanket depth(m)	0.99	0.43
Design Volume (m^3)	4284	11273.44
Effective Volume (m^3)	2870.28	8041.72

Below are results for the hydraulic retention time (HRT) together with the volumetric and surface loadings of the anaerobic and facultative ponds. The equations 2, 3 and 4 were used to obtain the HRTs, volumetric loading and surface loading respectively. These results are shown in table 4.6 along with the design values and recommended values.

$$Hydraulic\ Retention\ Time = \frac{Volume\ of\ pond}{Influent\ flow\ rate} \text{...Equation 2}$$

HRT (anaerobic pond) = 2870.28/3842.8 = 0.74 days

HRT (facultative pond) = 8041.72/2558 = 3.14 days

According to the HRT results obtained, the HRT for anaerobic ponds (0.74 days) is lower than the design HRT value of a minimum of 1 day. It is also less than the recommended HRT of 1-1.5 days which can explain the removal efficiencies of the anaerobic ponds generally being low. The facultative ponds are operating with a retention time of about 3 days which is slightly less than the design value of 3.5 days and is significantly lower than the recommended 2-3 weeks which could impede the formation of algae in the ponds. This can also explain why the values of chlorophyll-a obtained are much lower than the recommendation (Kayombo, *et.al*, 2004).

$$Volumetric\ loading = \frac{influent\ BOD \times Flow}{anaerobic\ pond\ volume} \quad \text{...Equation 3}$$

$$= (1830 \times 3842.8)/(2870 \times 2)$$

$$= 1225.1 g/m^3/d$$

From above, the volumetric loading for the anaerobic pond is 1225.1g/m^3/d which is considerably higher than the design volumetric loading of 340 g/ m^3/d and also higher than the recommended volumetric loading of 100-400 g/ m^3/d. This could be the cause of the odour nuisance at the anaerobic pond and shows possibility of failure to maintain anaerobic conditions in the ponds (WHO, 1987).

$$Surface\ loading = \frac{10 \times influent\ BOD \times Flow}{facultative\ pond\ area} \quad \text{...Equation 4}$$

$$= (10 \times 1545 \times 1585)/\ (8500 \times 2) = 1440.5\ kg\ BOD_5/ha/d$$

Clearly, the surface loading of the facultative pond is larger than the design loading of 331 kg BOD_5/ha/d which could impair the development of a healthy algal population required for an efficient facultative pond (Kayombo, *et.al*, 2004).

Table 4.6: measured, design and recommended values

	Measured value	Design value	Recommended value
HRT (anaerobic pond) in days	0.31	1	1-1.5
Volumetric loading (g/m^3/d)	1225.1	340	100-400
HRT (facultative pond) in days	3.14	3.5	14-21
Surface loading (kg BOD_5/ha/d)	1440.5	331	100-400

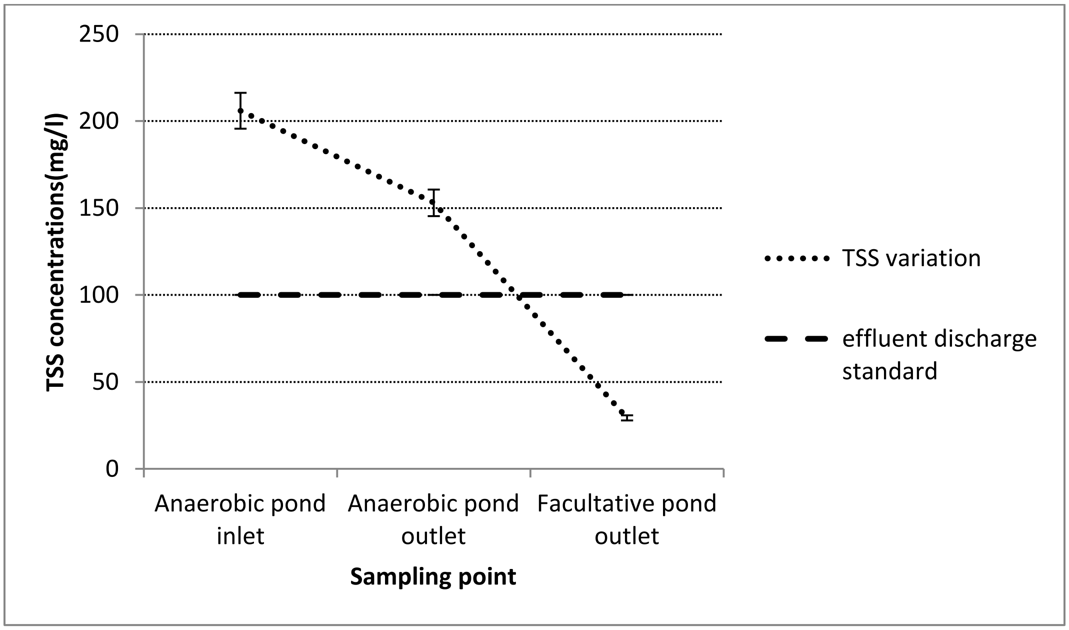
4.4 Performance Efficiency of the WSPs

4.4.1 Chlorophyll-a

The five days of sampling revealed an average effluent chlorophyll-a concentration of 318.4±26.2 µg/l which is well below the 1000-3000µg/l specified for efficiently operating facultative ponds by WHO (1987). This is likely due to the high organic loading received by the facultative pond (over 3000 kg.BOD/day). It is therefore possible that there is not sufficient oxygen produced for the aerobic bacterial oxidation of non-settleable organic compounds and the solubilized products of anaerobic digestion in the facultative pond (WHO, 1987). Raw data is located in the appendix. According to Mara, (2003), chlorophyll-a absorbs light energy which it uses to fix carbon dioxide and to produce oxygen which is necessary for the breakdown of organic compounds in the ponds. Therefore the concentration of chlorophyll-a represents the amount of oxygen in the ponds

4.4.2 Total Suspended Solids (TSS)

Figure 4.10 shows the variation of TSS concentrations from the anaerobic pond inlet to the facultative pond outlet. The average removal efficiencies of the anaerobic and facultative ponds were 78.5±7.7% and 93.8±5.8% respectively (Figure 4.10). However analysis after day 2 of sampling recorded a higher effluent TSS concentration than the influent of the anaerobic pond. This could be due to low HRT calculated for the ponds that could imply little time for the settlement of the suspended solids because of short circuiting. Desludging and re-positioning of inlet-outlet increases the removal efficiency of both BOD_5 and TSS. Desludging can therefore significantly improve the TSS removal as it is reported that the ponds are with a had high sludge accumulation (Gopolang & Letshwenyo, 2018). The average value of the effluent from the facultative pond was below the discharge standard of 100 mg/l implying that the system is efficient in the removal of total suspended solids.



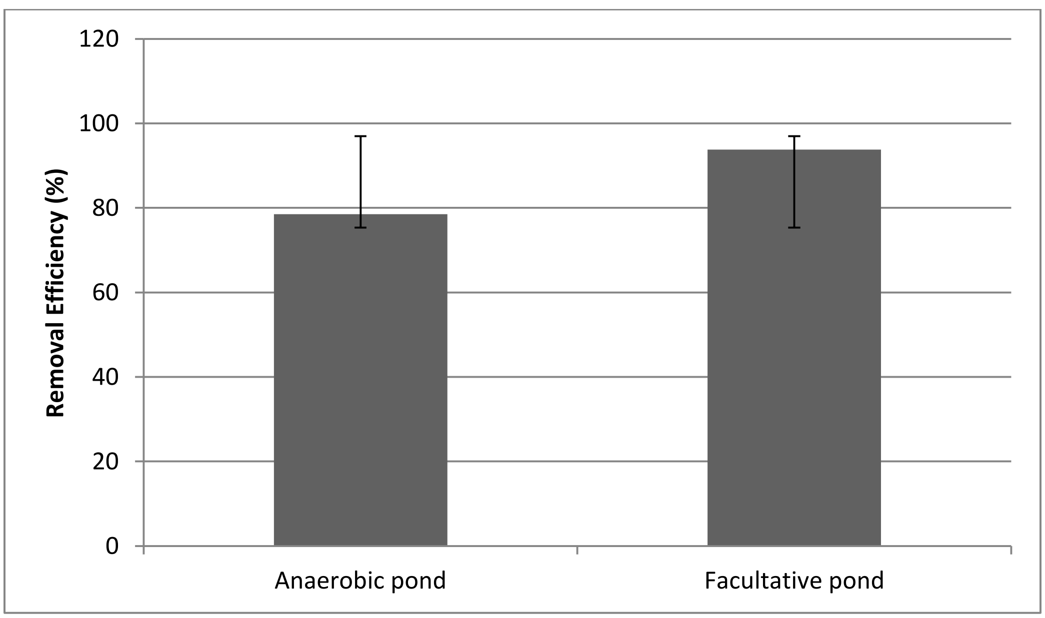
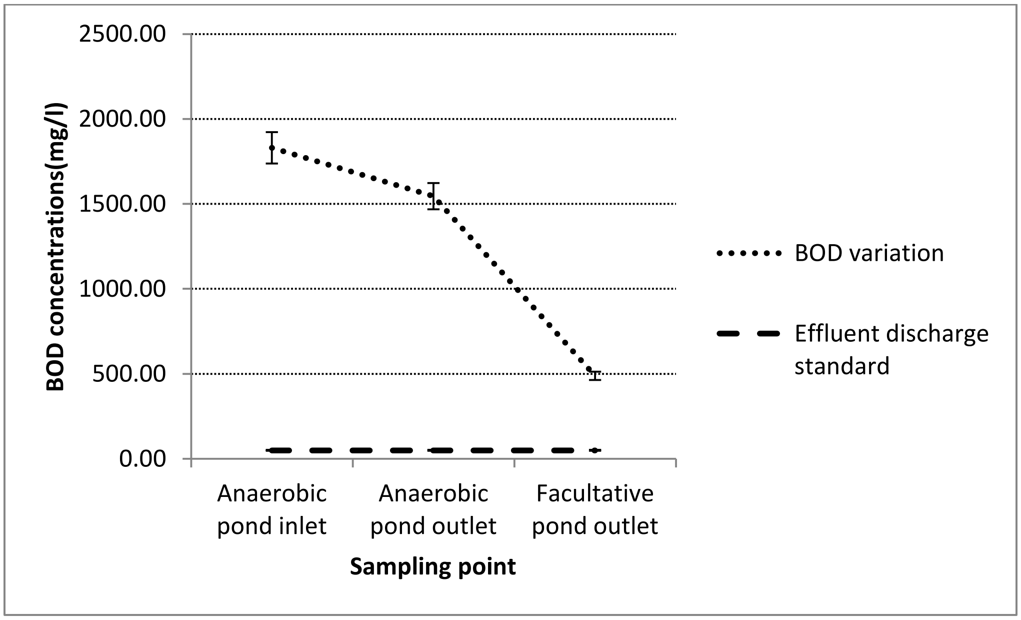


Figure 4.10: Total Suspended Solids concentrations variations and removal efficiencies

4.4.3 Biochemical Oxygen Demand (BOD_5)

The average removal efficiencies for the anaerobic and facultative ponds were 76.2±4.1% and 91.5±7% respectively. These met the design expected BOD_5 removal efficiencies The effluent from the facultative ponds was still with a BOD_5 concentration above the discharge standard i.e. 489 mg/l >50 mg/l. This could be due to the effective hydraulic retention time (3 days) being lower than the design HRT (3.5 days) which leads to less contact time for microbes to degrade the BOD_5. The conditions may not be ideal in the facultative ponds because of algae growth observed which could have increased BOD_5 concentration during die-offs and decaying at the bottom of the ponds (Gopolang & Letshwenyo, 2018). The BOD_5 concentration reduction trend and the removal efficiencies for the ponds are shown in Figure 4.11.



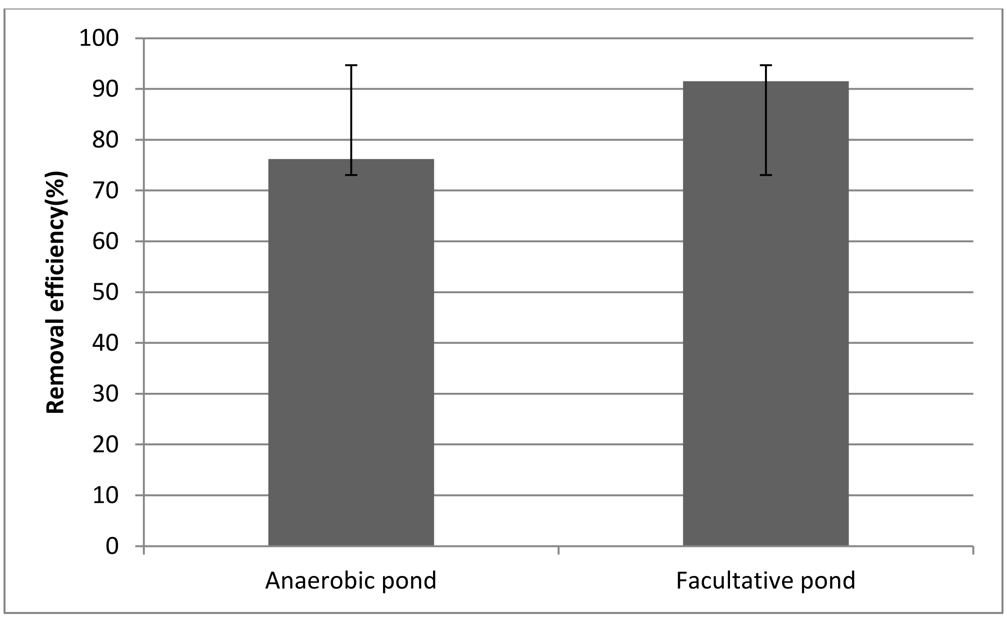


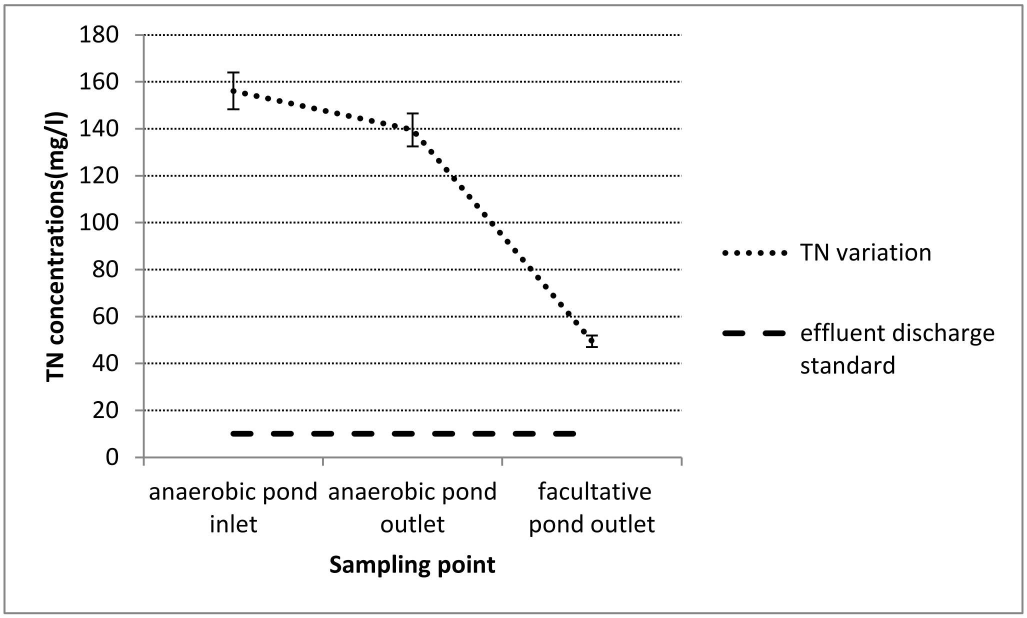
Figure 4.11: BOD concentration variations and removal efficiency

4.4.4 Total Nitrogen (TP) and Total Phosphorous (TP)

The average TN removal efficiency was about 74.8± 4.7% for anaerobic ponds and about 88.8± 12.5% for facultative ponds. There was an exception on sampling day 2 at the anaerobic pond where the effluent TN concentration was higher than the influent concentration which may be attributed to algae die off which increased the nitrates in the pond. The increase could also be due to the breakdown of organic matter being oxidised into nitrates. These could be the reasons the facultative pond effluent is still above the discharge standard of 10 mg/l. (Gopolang &

Letshwenyo, 2018). Figure 4.12 shows the variation of TN concentration along the treatment system together with the pond removal efficiencies.

The average TP removal efficiencies were 77.96±4.5% and 82.4±23.9% for anaerobic and facultative ponds respectively. The trend of total phosphorous concentrations and the removal efficiencies are illustrated in Figure 4.13 .



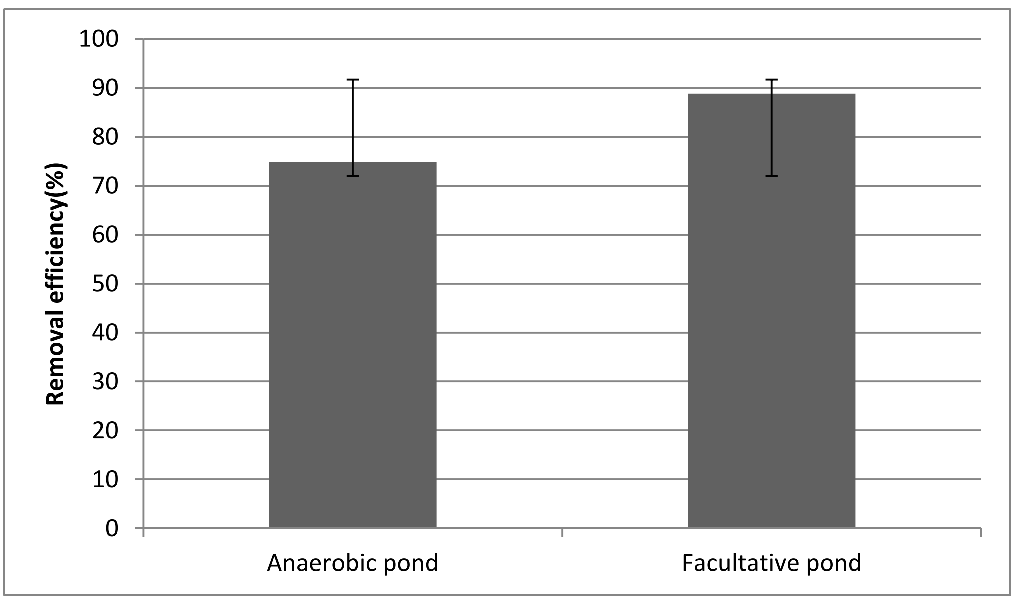


Figure 4.12: Total Nitrogen concentration variations and removal efficiencies

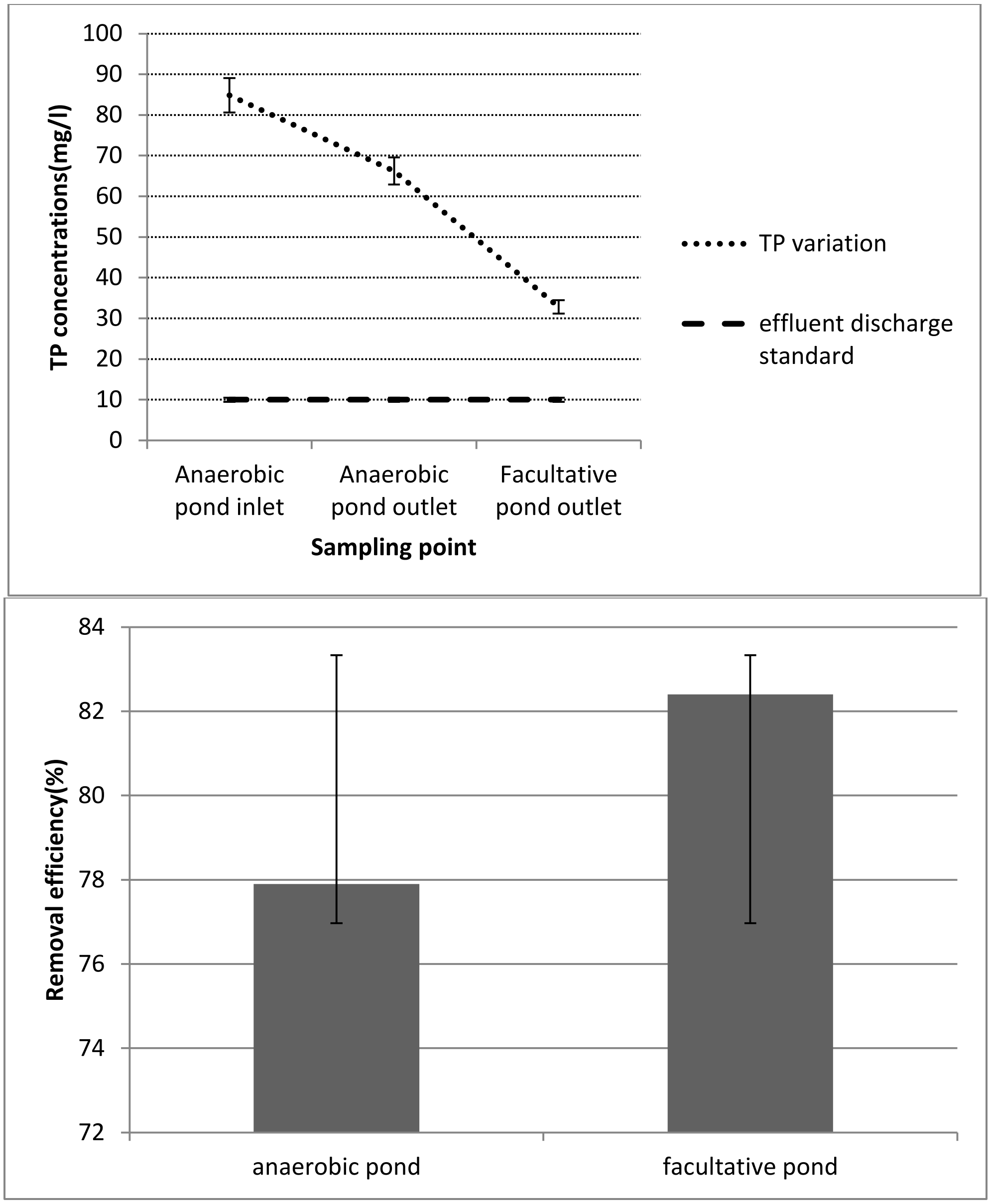


Figure 4.13: Total Phosphorous concentrations variations and removal efficiencies

4.4.5 Faecal Coliforms (FC)

The average removal efficiency of the facultative ponds was about 91.7± 12.4%. Day 1 revealed a certain increase in the FC concentration in the anaerobic pond. This could have been due to the nature of the pond being deep and therefore cause anaerobic conditions which enhance bacterial survival. It could also probably be due to the effective HRT (0.31 days) being lower than the design HRT of a minimum of 1 day causing low coliform removal. It could also be due to increased inflow and high organic loading on the ponds and high sludge blanket height in the ponds (Gopolang & Letshwenyo, 2018). This could explain why the facultative pond effluent is above the discharge stand for FC of 5000 cfu/100ml. The FC concentration variation is shown in Figure 4.14.

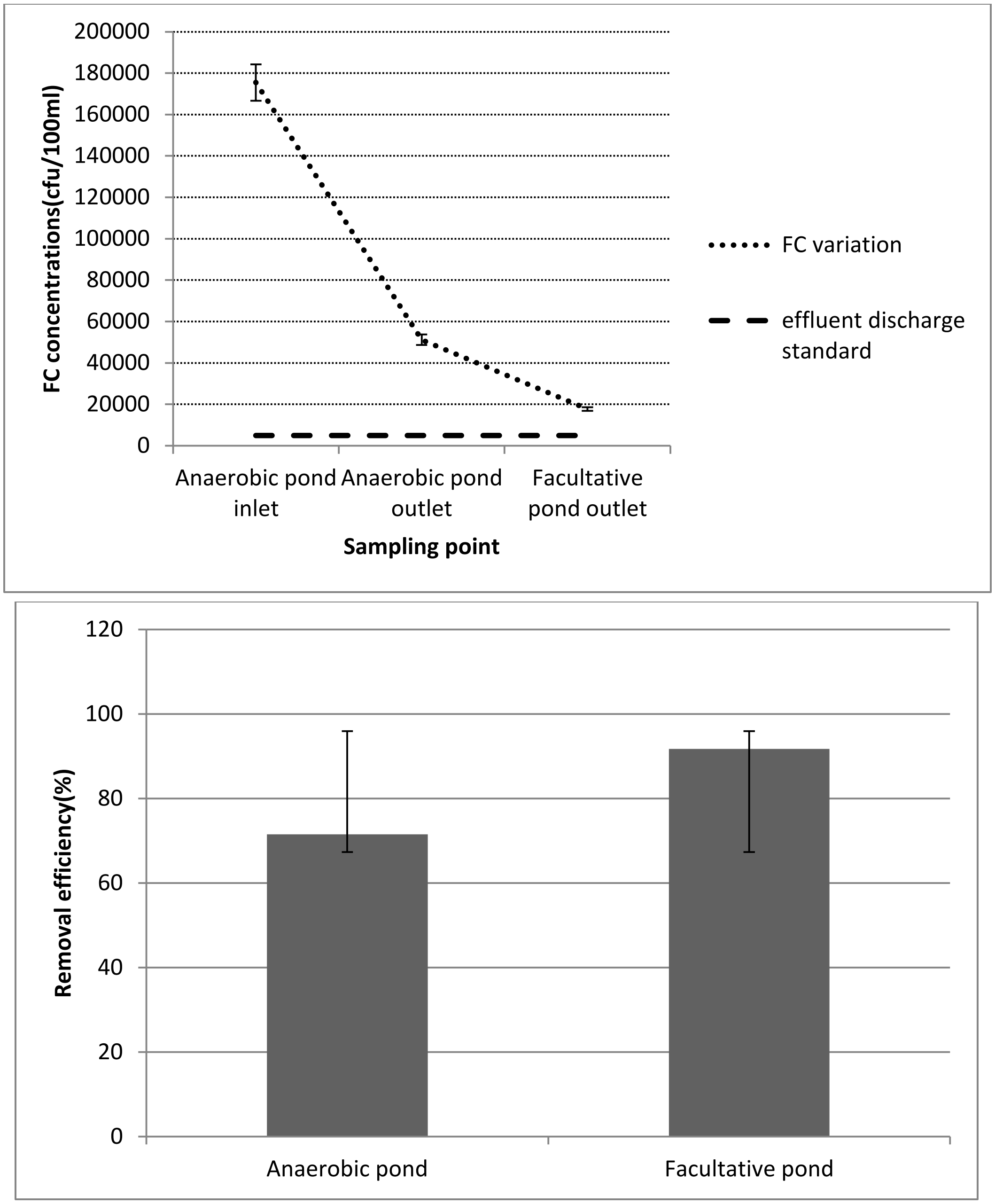


Figure 4.14: Faecal coliforms concentrations variations

4.5 Optimization of treatment processes

Below are the suggested methods to improve the performance of the Lubigi sewage and faecal sludge treatment plant;

Increasing the width of the facultative ponds by 7 metres in order to reduce the length to width ration to less than 3 as recommended by WHO (1987). The design report provided a L:W ratio of 3.4:1 for the facultative ponds. Since there is likely sludge accumulated around the inlet to the facultative pond, increasing the width will reduce the length to width ratio of the ponds and thereby reduce this effect.

From the study, it was noticed that the facultative ponds were with higher sludge depths nearer to the inlet than towards the outlet. Since the design was made for only one inlet to the pond, providing multiple inlets (well separated from the outlet) to the ponds will disperse effluent within the pond and allow sludge to deposit more evenly within the pond (Australian Pork Limited, 2015).

Measurements of sludge depths indicated high sludge blanket heights especially at the anaerobic ponds. This could mean that there is a delay in desludging of the ponds or there is need to increase the frequency of the desludging due to higher loadings received by the ponds. This will increase the effective volume of the pond and thus reduce the volumetric loading thereby improving the performance. It would also increase the hydraulic retention time of the ponds.

Increasing the facultative pond area will reduce on the BOD surface loading of the facultative pond and likely improve performance. It could reduce the measured surface loading of 1440.5kgBOD_5/ha/d closer to the design loading of 331kgBOD_5/ha/d.

Increasing the volume of the anaerobic pond will reduce the measured volumetric loading from 1225.1g/m^3/d to the design value of 340g/m^3/d thereby improving the performance of the treatment system.

Reducing the amount of FS allowed into the treatment system to reduce the organic load on the plant thereby reducing odour problems experienced at the anaerobic ponds and improve the growth of algae at the facultative ponds. The design was made for a maximum of 400m^3/day of faecal sludge flow. However on some sampling days, take for example 19/02/2020; the FS flow was measured to be up to 518m^3/day which is significantly higher than the design flow.

CHAPTER FIVE - CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The following was drawn from the study with respect to the objectives after analysis and discussion of the results;

- The characteristics of the influent WW were as follows; the average flow rate was in the the range of 2845m^3/day to 4057m^3/day with a median 3433.6 m^3/day. The BOD concentration ranged from 209 to 623 mg/l with a mean value of 412.6 mg/l. The TSS concentration ranged between 271 and 392 mg/l with a mean value of 336.9 mg/l. The TN concentration varied from 70 to 282 mg/l with an average value of 196.1 mg/l. The concentration of TP ranged from 15 to 85 mg/l with a mean value of 56.9 mg/l. The average value of the FC concentration was 2.373×10^6 cfu/100ml ranging between 4.64×10^5 and 4.9×10^6 cfu/100ml.
- The effluent from the FS thickening tanks had the following characteristics; and the average flow rate was in the range of 138.2 m^3/day to 518.4 m^3/day with a median of 414.7 m^3/day. The BOD ranged from 1002 to 1622 mg/l with a mean value of 1344.8 mg/l. The TSS concentration ranged between 201 and 322 mg/l with an average value of 245.5 mg/l. The concentration of TN ranged from 130 to 311 mg/l with a mean value of 248.7 mg/l. The average TP concentration was 95.4 mg/l ranging between 77 and 122 mg/l. The FC concentration varied from 5.1×10^5 to 1.372×10^6 cfu/100ml with a mean value of 9.362×10^5 cfu/100ml.

- The inlet and outlet positions and depths were as recommended. The length to width ratios were as recommended for the anaerobic ponds but were slightly higher than recommended for the facultative ponds. Due to the sludge accumulation in the ponds, the retention times were found to be lower than recommended with the anaerobic ponds and facultative ponds having HRTs of 0.74 days and 3.14 days respectively. The volumetric and surface loadings were found to be higher than recommended i.e. 1225.1 g/m^3/d and 1440.5 kg/ha/d respectively.

- An average chlorophyll-a concentration of 318.4±26.2 µg/l was observed at the facultative ponds. . The average TSS removal efficiencies of the anaerobic and facultative ponds were 78.5±7.7% and 93.8±5.8% respectively. The average value of the effluent from the facultative pond was below the discharge standard of 100 mg/l. The average BOD removal efficiencies of the anaerobic and facultative ponds were 76.2±4.1% and 91.5±7% respectively. The effluent from the facultative pond was still above the discharge standard of 50mg/l. The average TN removal efficiency was about 74.8±4.7% for anaerobic ponds and about 88.8±12.5% for facultative ponds. The average TP removal efficiencies were 77.96±4.5% and 82.4±23.9% for anaerobic and facultative ponds respectively. The TN and TP concentrations for the effluent from the facultative ponds were above the discharge standard of 10 mg/l. The average FC removal efficiencies for the anaerobic ponds and facultative ponds were 71.5±6.6% and

91.7±12.4% respectively and the effluent from the facultative ponds was still above the discharge standard of 5000 mg/l.

5.2 Recommendations

Given the conclusions made in 5.1, the following recommendations are suggested;

- Providing multiple inlets to the ponds.
- Increasing the desludging frequency of the ponds.
- Reducing the FS load on the plant.
- Increasing the area of the facultative ponds.
- Increasing the volume of the anaerobic ponds

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APPENDIX 1

Table 1

	Anaerobic Pond													
	Concentrations													
Date	Inflow(m^3/d)	Outflow(m^3/d)	Input BOD (mg/l)	Output BOD (mg/l)	Input TSS (mg/l)	Output TSS (mg/l)	Input TN (mg/l)	Output TN (mg/l)	Input TP (mg/l)	Output TP (mg/l)	Input FC (CFU/100ml)	Output FC (CFU/100ml)	Inflow (ft^3/s)	Outflow (ft^3/s)
13/02/2020	9092.3169	2143.1436	2146.00	1681.14	198.44	105.33	171.37	141.44	101.44	72.11	9900	44000	3.71634	0.875977
19/02/2020	11,161.94	3361.6456	1569.46	1409.06	161.51	164.51	142.5	144.6	73.41	66.51	161000	51000	4.56227	1.374021
25/2/2020	6614.3782	1984.43	1810.5	1363.5	298.11	221.21	144.67	134.1	69.49	47.99	220,000	19,000	2.70352	0.811105
2/3/2020	8926.2935	2305.8721	2066.8	1758.64	197.23	114.21	177.43	145.91	103.91	77.32	172000	49000	3.64848	0.94249
13/3/2020	10133.804	2995.1342	1557.11	1512.2	174.37	159.6	144.6	131.61	75.83	67.22	149000	93000	4.14204	1.224215

Table 2

	Anaerobic Pond														
	Loads										REMOVAL EFFICIENCIES				
Date	Input BOD (kg/d)	Output BOD (kg/d)	Input TSS (kg/d)	Output TSS (kg/d)	Input TN (kg/d)	Output TN (kg/d)	Input TP (kg/d)	Output TP (kg/d)	Input FC (CFU/day)	Output FC (CFU/day)	BOD (%)	TSS (%)	TN (%)	TP (%)	FC (%)
13/02/2020	19512.11	3602.9244	1804.279	225.7373	1558.15	303.126	922.325	154.542	9E+11	9.4E+11	81.5349	87.4888	80.5458	83.24429	-4.75969
19/02/2020	17518.217	4736.7603	1802.765	553.0243	1590.576	486.094	819.398	223.583	1.8E+13	1.7E+12	72.9609	69.3235	69.4391	72.71374	90.4598
25/2/2020	11975.332	2705.7703	1971.812	438.9758	956.9021	266.112	459.633	95.2328	1.5E+13	3.8E+11	77.4055	77.7374	72.1903	79.28069	97.4089
2/3/2020	18448.863	4055.199	1760.533	263.3537	1583.792	336.45	927.531	178.29	1.5E+13	1.1E+12	78.0192	85.0413	78.7567	80.778	92.6408
13/3/2020	15779.448	4529.242	1767.031	478.0234	1465.348	394.19	768.446	201.333	1.5E+13	2.8E+12	71.2966	72.9477	73.0993	73.8	81.5524

Table 3

	Facultative Pond													
	Concentrations													
Date	Inflow(m³/d)	Outflow(m³/d)	Input BOD (mg/l)	Output BOD (mg/l)	Input TSS (mg/l)	Output TSS (mg/l)	Input TN (mg/l)	Output TN (mg/l)	Input TP (mg/l)	Output TP (mg/l)	Input FC (CFU/100ml)	Output FC (CFU/100ml)	Inflow (ft³/s)	Ouflow (ft³/s)
13/02/2020	2143.1436	726.50829	1681.14	786.26	105.33	32.14	141.44	51.32	72.11	32.34	44000	120	0.87598	0.296949
19/02/2020	3361.6456	139.81658	1409.06	287.9	164.5	17.52	144.6	33.4	66.51	25.25	51000	21000	1.37402	0.057148
25/2/2020	1984.43	1118.5327	1363.5	323.18	221.21	52.3	134.1	75.69	47.99	50.31	19000	10,000	0.81111	0.457183
2/3/2020	2305.8721	586.16342	1758.64	768.68	114.21	29.15	145.91	55.11	77.32	29.94	49000	17000	0.94249	0.239585
13/3/2020	2995.1342	174.681	1512.2	279.42	159.6	15.31	131.61	31.6	67.22	26.44	93000	23000	1.22421	0.071398

Table 4

	Facultative Pond														
	Loads										REMOVAL EFFICIENCIES				
Date	Input BOD (kg/d)	Output BOD (kg/d)	Input TSS (kg/d)	Output TSS (kg/d)	Input TN (kg/d)	Output TN (kg/d)	Input TP (kg/d)	Output TP (kg/d)	Input FC (CFU/day)	Output FC (CFU/day)	BOD (%)	TSS (%)	TN (%)	TP (%)	FC (%)
13/02/2020	3602.9244	571.22441	225.7373	23.34998	303.1262	37.2844	154.542	23.4953	9.4E+11	8.7E+08	84.1455	89.6561	87.7	84.79684	99.9075
19/02/2020	4736.7603	40.253193	552.9907	2.449586	486.094	4.66987	223.583	3.53037	1.7E+12	2.9E+10	99.1502	99.557	99.0393	98.421	98.2874
25/2/2020	2705.7703	361.48739	438.9758	58.49926	266.1121	84.6617	95.2328	56.2734	3.8E+11	1.1E+11	86.6401	86.6737	68.1857	40.90966	70.334
2/3/2020	4055.199	450.5721	263.3537	17.08666	336.4498	32.3035	178.29	17.5497	1.1E+12	1E+11	88.889	93.5119	90.3987	90.15664	91.1807
13/3/2020	4529.242	48.809365	478.0234	2.674366	394.1896	5.51992	201.333	4.61857	2.8E+12	4E+10	98.9224	99.4405	98.5997	97.70601	98.5576

Table 5

	Influent WW Loads				
Date	BOD(kg/d)	TSS (kg/d)	TN (kg/d)	TP (kg/d)	FC (CFU/day)
13/02/2020	2208.075149	2254.071	1283.37	366.3196	2.21751E+14
19/02/2020	5137.835801	3308.219	2328.541	714.1987	3.92175E+13
25/2/2020	1239.955127	1607.424	415.9073	91.36324	2.90512E+14
2/3/2020	2156.592493	2253.721	1190.917	356.4339	1.9836E+14
13/3/2020	4768.458709	2893.97	2156.758	625.8516	3.83971E+13

Table 4.2: Characteristics of raw wastewater and effluent from FS thickening tanks

	Influent wastewater		**FS thickening tanks Output**	
Parameter	Average	Range	Average	Range
TSS (mg/l)	336.9±48.7	271.1 – 391.4	245.5±52.6	201.5 - 321.6
BOD (mg/l)	412.6±189.6	209.1 - 622.2	1344.8±312.7	1002.9 - 1621.5
TN (mg/l)	196.1±86.9	70.2 - 281.4	248.7±72.9	130.2 – 311.4
TP (mg/l)	56.9±28.1	15.4 - 84.5	95.4±21.1	77.8 – 121.5
FC (cfu/100ml)	$2.373\times10^{6}\pm1.893\times10^{6}$	4.64×10^{5}- 4.9×10^{6}	9.362×10^{5} $\pm3.618\times10^{5}$	5.1×10^{5}-1.372×10^{6}

Table 6

	Effluent from FS thickening tanks Loads				
Date	BOD(kg/d)	TSS (kg/d)	TN (kg/d)	TP (kg/d)	FC (CFU/day)
13/02/2020	1948.6911	390.7349	475.2768	157.6823	1.43488E+13
19/02/2020	4206.0099	871.4985	843.8577	329.3592	3.71795E+13
25/2/2020	1111.623	144.5761	89.22463	53.34968	3.49631E+12
2/3/2020	2092.232	446.9615	524.5079	169.8429	1.68987E+13
13/3/2020	3809.7432	690.2782	754.7371	284.3107	3.08464E+13

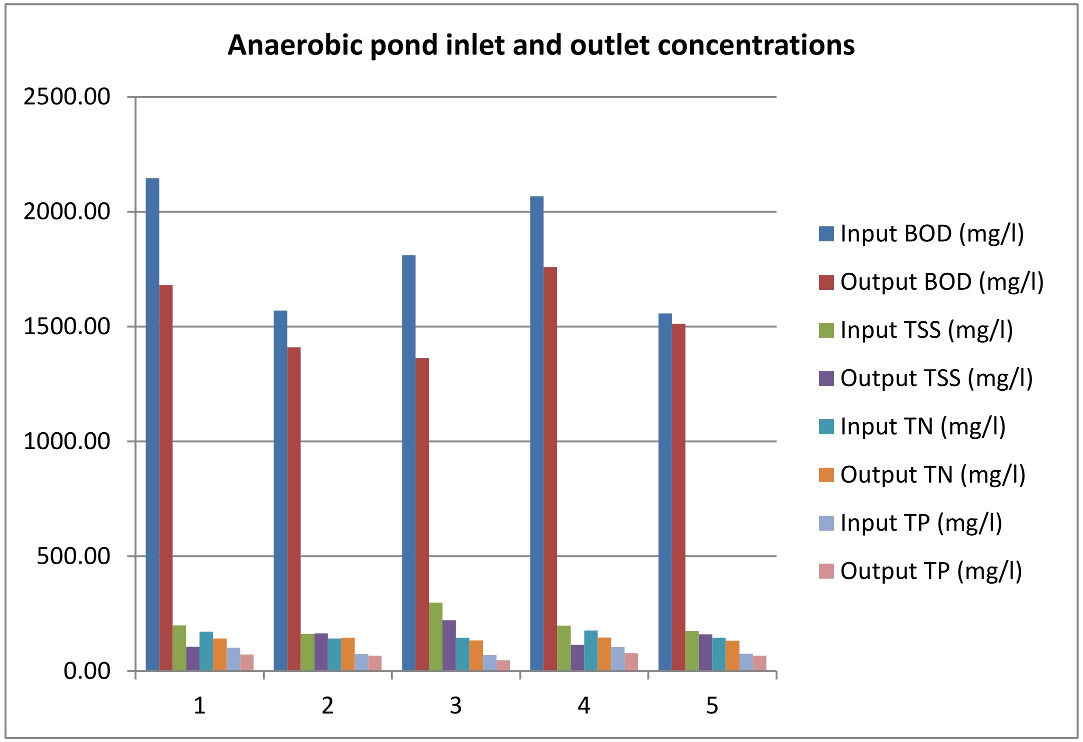


Figure 1

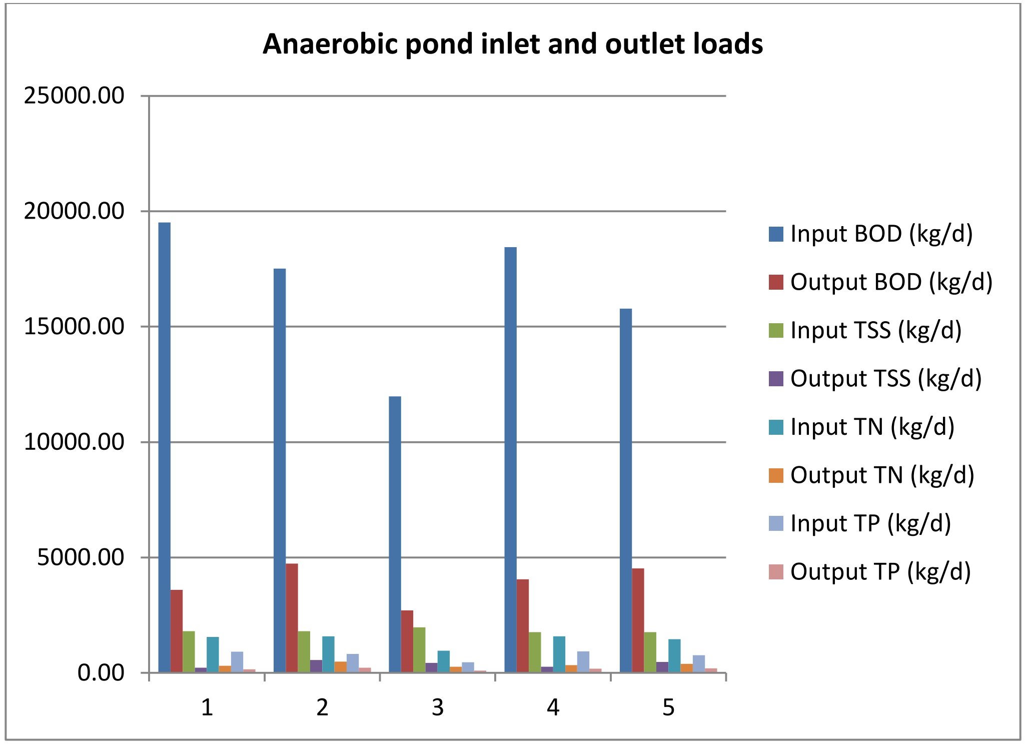


Figure 2

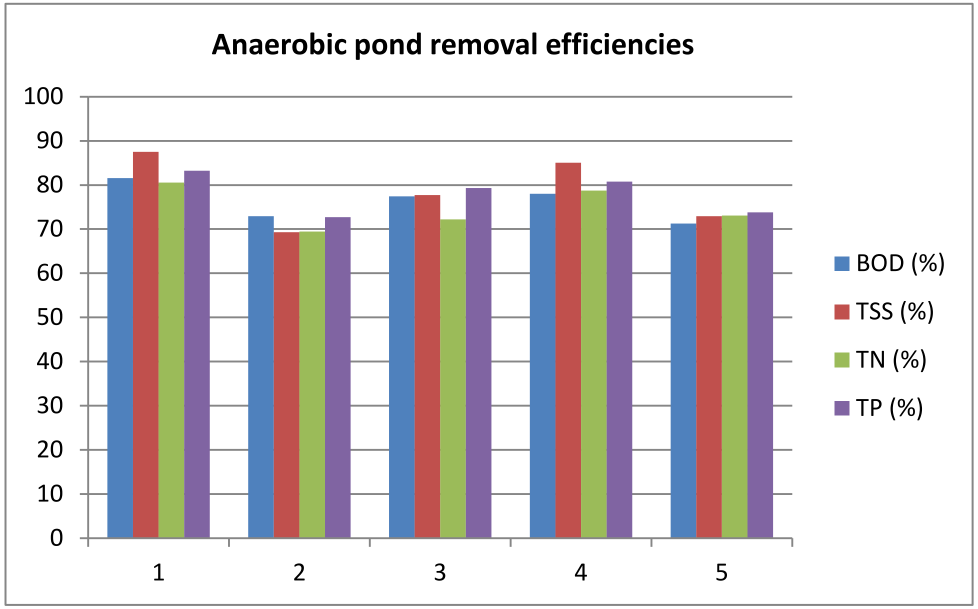


Figure 3

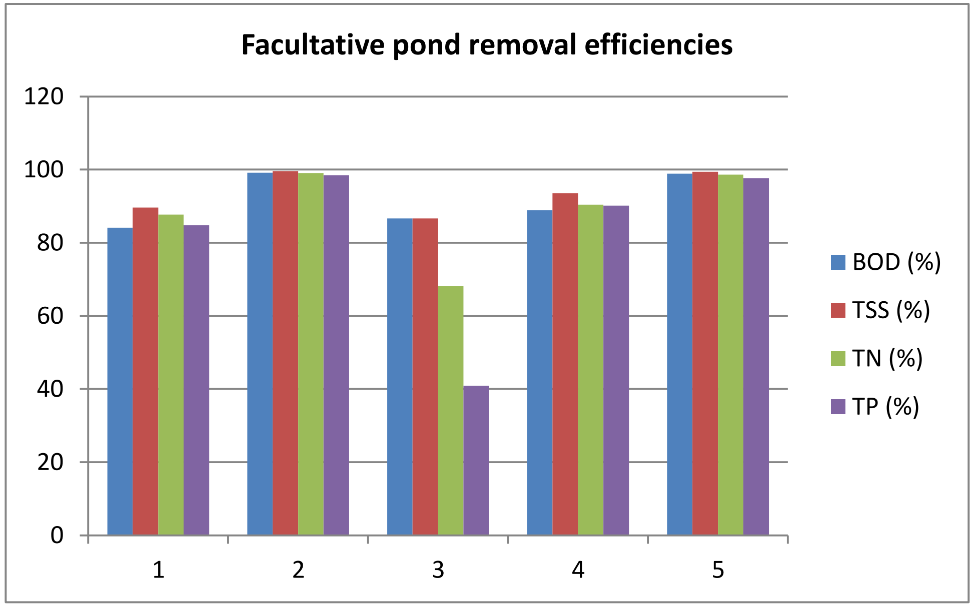


Figure 4

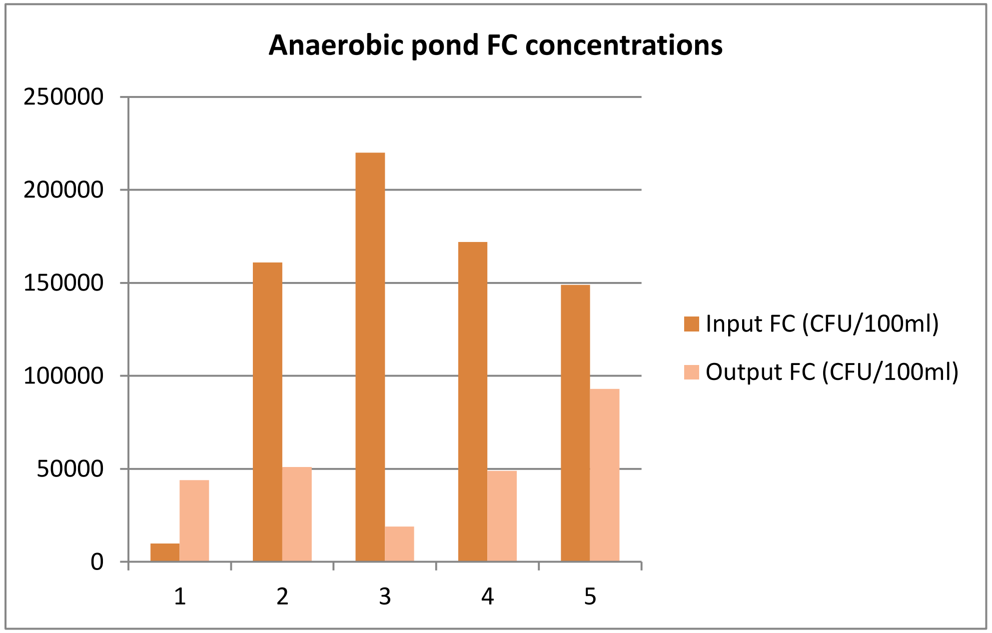


Figure 5

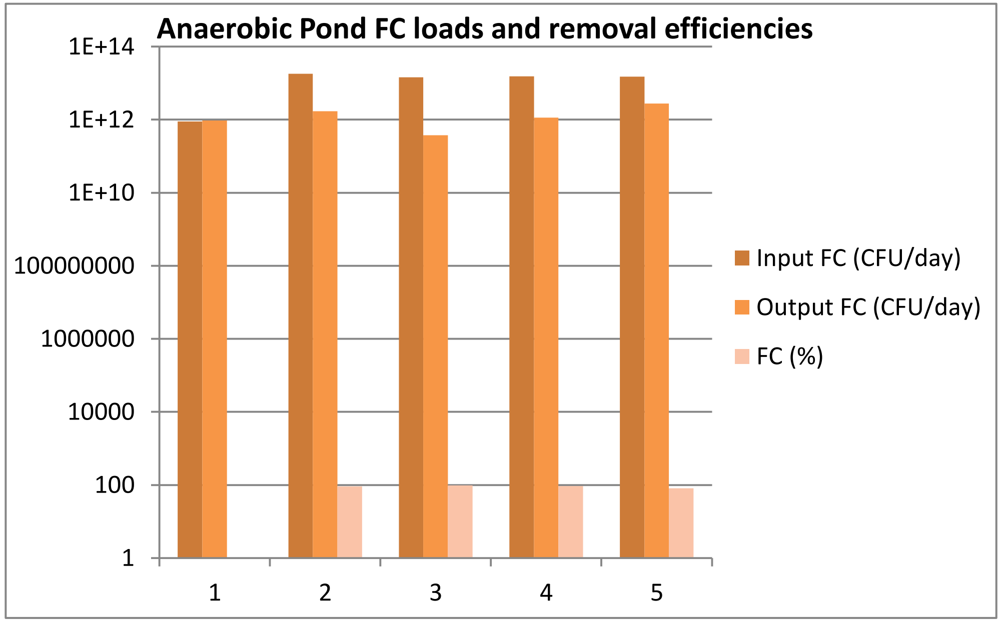
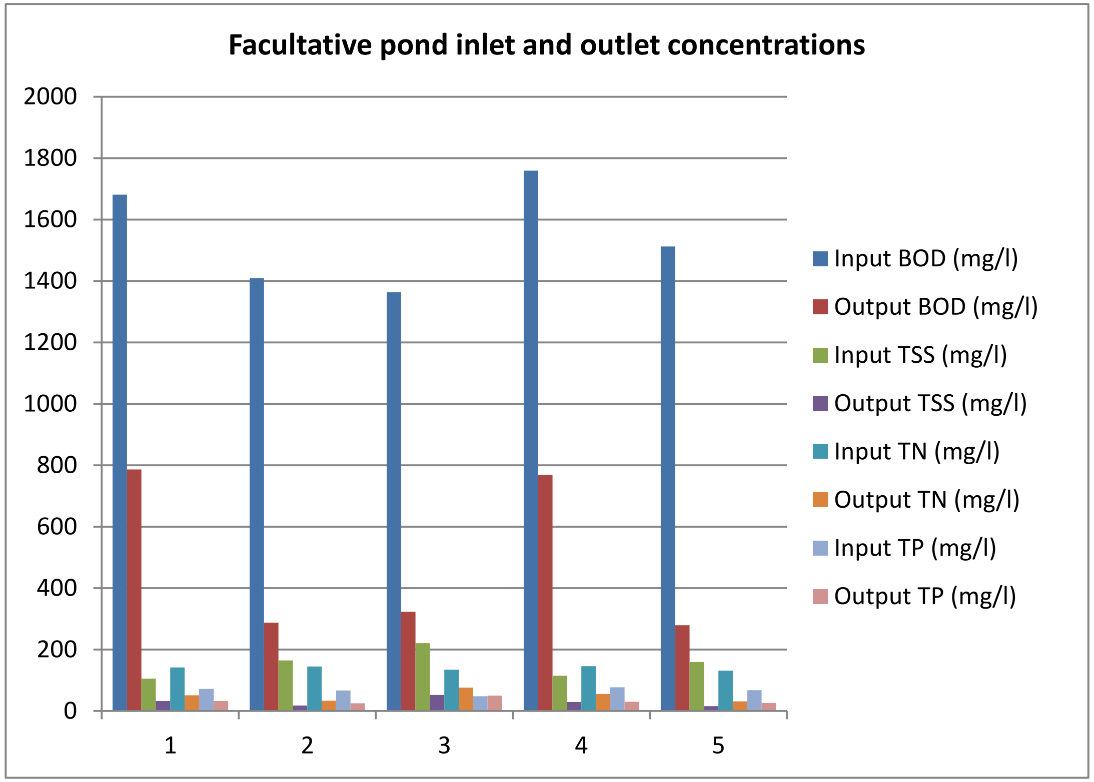


Figure 6



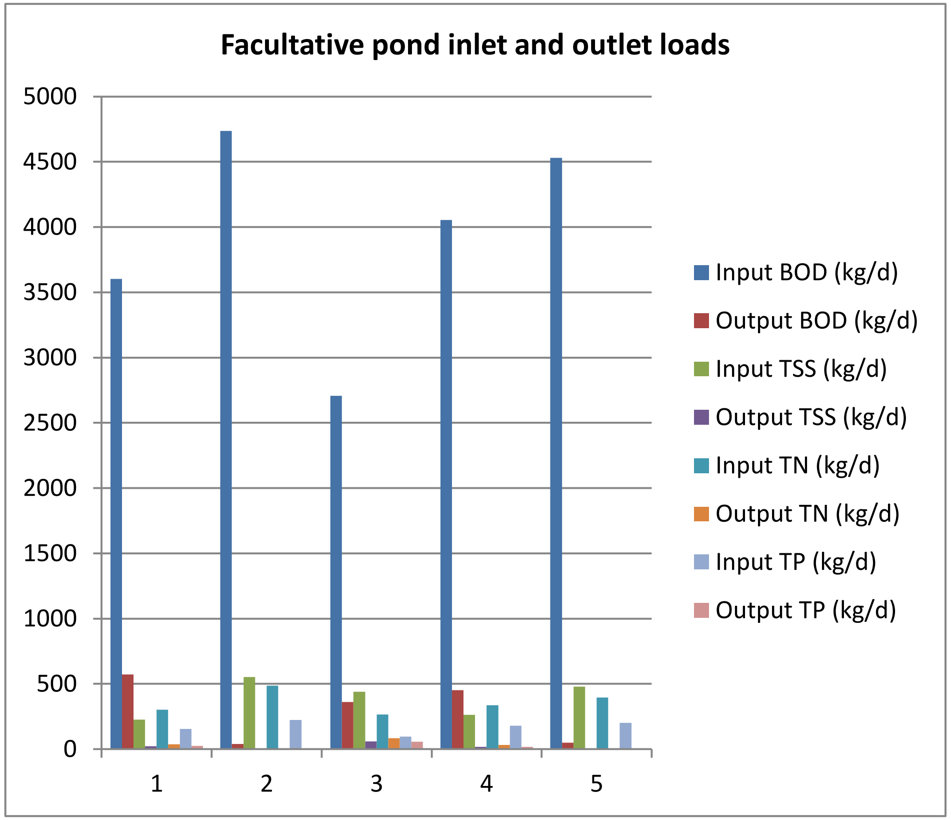


Figure 7

Figure 8

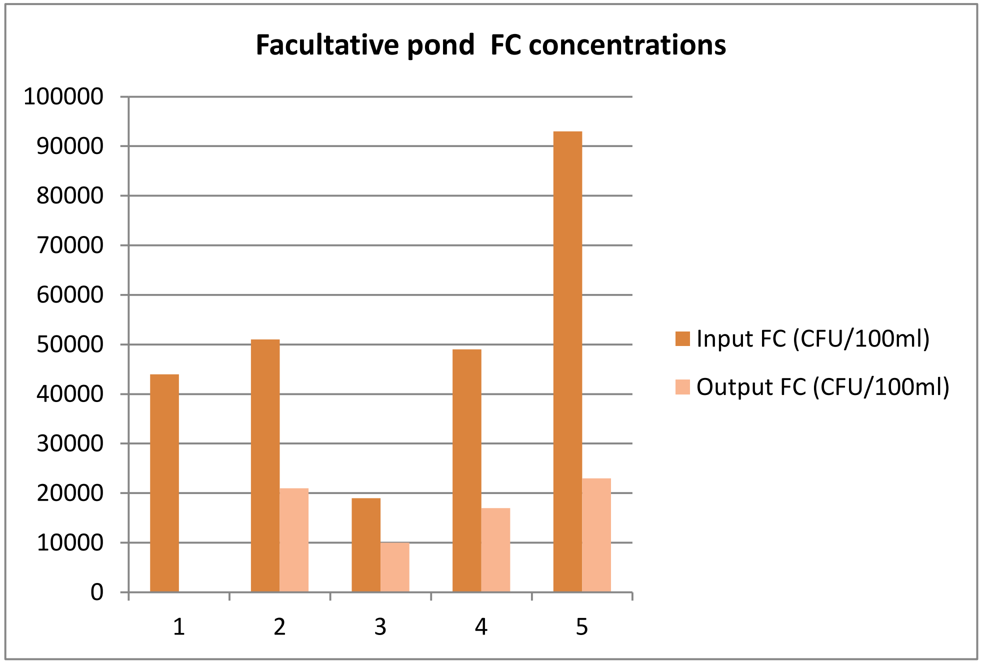


Figure 9

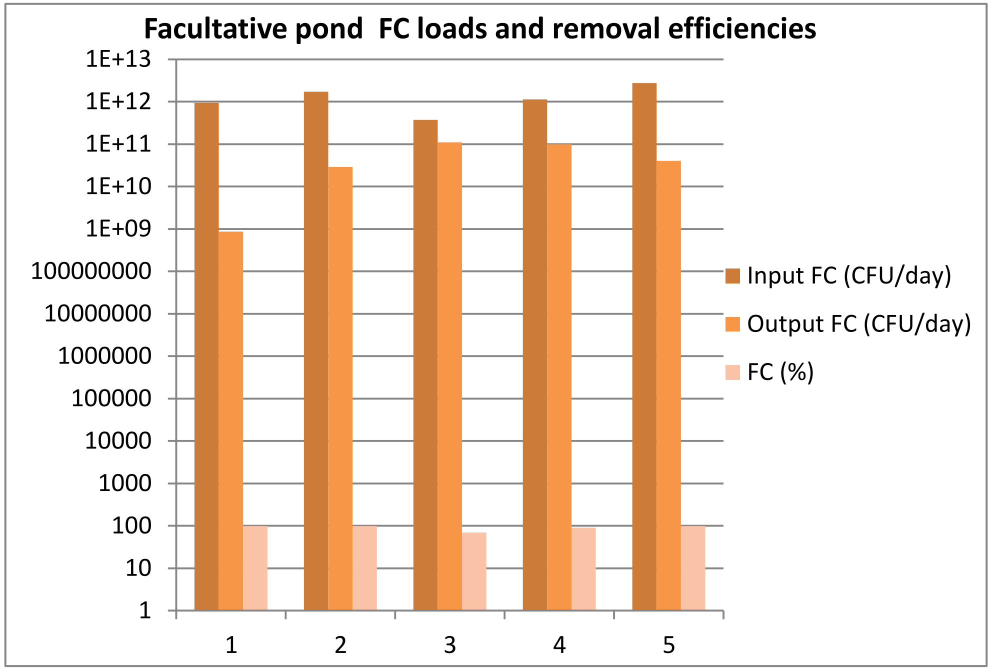


Figure 10