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**Evaluation of vertical subsurface flow constructed wetland system
for polluted River wastewater treatment: a case study on Akaki
river source of urban agriculture in Addis Ababa.**

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DECLARATION

I the undersigned, declare that this thesis is my own original work performed under the supervision of my research advisor Dr Alemtsehay Gebremeskel, school of civil and Environmental Engineering, Addis Ababa University as part of the Degree of Master of science in water supply Engineering Programing accordance with the rule and regulation of the institute and all sources of materials used for this thesis have also been duly acknowledged.

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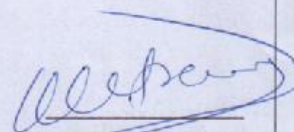
**Study on the Evaluation of Vertical Subsurface flow
constructed wetland system for polluted River wastewater
treatment: a case study on Akaki River source of urban
agriculture in Addis Ababa.**

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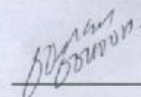


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ABSTRACT

Urban Agriculture (UA) in Addis Ababa has a number of challenges and opportunities since Urban farmers on the riverbank mainly dependent on irrigated water from Akaki River. A number of studies have been conducted focused on characterization of Akaki River water quality, challenges faced by farmers in UA, health risks associated with irrigated vegetables by the river and economic losses resulted due to the river pollution. However a realistic solution to the urgent situation of UA was not given yet. This study aims at performance evaluation of Vertical subsurface flow constructed wetland (VSSFCW) as cheap and efficient onsite wastewater treatment solution functioned by farmers. No odors or insects were detected during the six month operation of VFCW, beside a beautiful landscape view. The efficiency of VSSFCW was studied on pilot plant. An evaluation of the efficiency in the purification of the main polluting agents were done. The average removal efficiencies of chemical oxygen demand (COD), biological oxygen demand (BOD), total suspended solids (TSS), Turbidity, Facial coliform(FC), Sulfide(S_2), Total ammonia(NH_3), nitrate(NO_3), Nitrite(NO_2), total nitrogen(T-N), total phosphorous(T-P), were 76 %, 87 %, 93 %, 100%, 94 %, 80 %, 16.7 %, 97.7 %, 43 %, 46 % respectively. The physiochemical characters of treated wastewater were complying with the WHO standard for wastewater reuse in Irrigation of crops likely to be eaten uncooked. However biological parameters were slightly above the guidelines.

ABERVATIONS

AAWSA	Addis Ababa Water Supply and Sewerage Authority
APHA	American Public Health Association
ASM 1	Activated Sludge Model 1
BOD5	Five day Biochemical Oxygen Demand
CFU	Colony Forming Unit
COD	Chemical Oxygen Demand
CW	Constructed Wetland
CWM1	Constructed Wetland Model No.1
CW2D	Constructed wetland two dimensional model
DF	Down flow
EEPA	Ethiopian Environmental Protection Authority
EIBC	Ethiopian Institute of Biodiversity Conservation.
ESTA	Ethiopian Science and Technology Agency
FC	Fecal Coliforms
HR	Horizontal Flow
HRT	Hydraulic Retention Time
IUCN	International Union for Conservation of Nature and Natural Resources
JWBO	Jehovah's Witnesses Branch Office
L	Litter
MF	Membrane Filter
ml	Milliliter

Mg	Milligram
M.a.s.l	Meters above sea level
SF	Surface Flow
SRT	Sludge retention time
SSF	Subsurface Flow
STELLA	Structural Thinking Experimental Learning Laboratory with Animation
TC	Total Coli forms
TN	Total Nitrogen
TP	Total Phosphorous
TSS	Total Suspended Solids
UA	Urban Agriculture
UNDP	United Nations Development Program
USEPA	United States of America Environmental Protection Agency
UNIDO	United Nations Industrial Development Organization
VF	Vertical Flow
VSSFCW	vertical SubSurface Flow Constructed Wetland

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1. INTRODUCTION

1.1 Background

In 2008, for the first time ever, more than 50% of the world's population living in urban settings. The highest rates of urbanization currently occur in poor areas of Africa and Asia (Robert & Sally, 1998). Many of the countries in which, fast urban growths are least-developed nations. These countries are not capable enough to provide sufficient food demanded by the expanding urban population via imports from rural areas (Tewodros, 2007). According to FAO (2001), Urban Agriculture (UA) contributes to urban food security through increased food availability, stability and, to some extent accessibility. The disposal of waste has become a serious problem in many cities. UA can contribute to solve this problem by turning urban wastes into a productive resource through compost production, and irrigation with wastewater. UA and forestry can also have a positive impact upon the greening of the city. The improvement of the urban micro-climate (wind breaks, dust reduction, shade), the maintenance of biodiversity as well as the reduction of the ecological foot print of the city by producing fresh foods close to the consumers and thereby reducing post-harvest losses, energy use for transporting, packaging, cooling, etc. (Veenhuizen et al, 2007).

Despite its positive contribution to stimulate agricultural production, unwise use of wastewater for irrigation has associated adverse impacts on environment public and animal health. Wastewater mostly comprised of organic matters, nutrients, heavy metals, pathogens and other miscellaneous constituents (Sinshaw, 2011).

UA is a traditional practice in Ethiopia, and the urban-based population is used to keeping cattle, sheep, and chickens, or growing rain-fed crops and vegetables, on the plots adjacent to their houses (Gittleman, 2009). UA in Addis Ababa create large number of employment and source of income for the city residents. In spite of its benefits, UA in the study area has many challenges with associated effects and root causes. Urban farmers on the riverbank mainly dependent on irrigated water from Akaki River.

Akaki River is the most polluted river system in the country. Yohannes & Elias(2017) reported that water quality in almost all the assessed sites along the Akaki rivers was found to be deteriorated and not meeting the river water quality standard. Such deterioration derived from different sources of pollutants both point (factories discharge, urban wastewater discharges, garage wastes, hospital wastes, etc.) and non-point sources (e.g. different sewages runoff, agricultural runoff). Industrial wastes are the dominant sources.

Akaki River catchment aquifers undergo a serious pollution and proper attention is important. The continual development without intervention will be risky for the environment, crops, public and animal health. Moreover, farmers have insufficient capacity to manage wastewater at field level (Tadesse, 2011). Therefore, Low cost pollution control and wastewater treatment mechanisms such as buffer zone vegetation and construction of Wetland by using locally available materials are very important based on the country's economy.

The need to implement onsite treatment solutions is therefore great, and ecological technologies such as subsurface flow constructed wetlands (SSFCWs) are gaining interest as low-cost and efficient options (Steiner & Combs, 1993). In SSFCWs, the wastewater is passed through a filter bed consisting of a porous medium with submerged plants (Fonder & Headley, 2010), where attached microbes perform a large part of the treatment processes. Subsurface flow wetlands with horizontal flow (HF) have long been used, but an inherent problem has been the poor oxygenation of such systems limits aerobic treatment processes in particular nitrification. This study focuses on vertical flow (VF) SSF wetlands, which are now becoming more popular as they are more aerobic systems and therefore have a higher ability to remove ammonium and organic matter (Wissing & Hofman, 2002).

Environmentalists have referred to wetlands as nature's kidney. Much interest has developed in recent years in using CW to remove contaminants from water, whether it is effluent from domestic, municipal, industrial, agricultural wastewaters, or acid-mine drainage (Kenneth, 2001).

Constructed wetlands become popular in East Africa. The CW in Uganda used to treat Municipal Wastewaters, while in Tanzania, used to treat Institutional Wastewaters (Bojcevska, 2004;

Kaseva, 2004). Number of studies that were carried out concerning the performance of wetlands on the removals of nitrogen, pathogens, Total Suspended Solids (TSS), Biochemical Oxygen Demand (BOD), and Chemical Oxygen Demand (COD) (Bojcevska et al., 2004; Kaseva, 2004; Senzia et al., 2002). In Ethiopia, only very few institutions such as Jehovah's Witnesses Branch office has used the Horizontal subsurface constructed wetland system to treat its domestic wastewater.

The primary purpose of this study was to evaluate the effectiveness of Vertical Subsurface Flow Constructed Wetland (VSSFCW) to treat the non-point source polluted wastewater from Akaki River, which is the main source for the UA in the city of Addis Ababa.

1.2 Statement of the problem

Urban Agriculture (UA) in the city has a number of challenges and opportunities. In spite of its benefits, UA in the study area has many challenges with associated effects and root causes. Urban farmers on the riverbank mainly dependent on irrigated water from Akaki River. However, the overall classification of the water quality of the Akaki River is very bad. Parameters such as different ions, heavy metals, fecal coliforms were found as the major threats impairing the water quality for different uses such as irrigation, swimming and aquatic ecosystem preservation (Yohannes & Elias, 2017). For example according to WHO standards for agricultural use, the maximum tolerable limit of total coliforms and E.colis for raw eaten and other crops is 200 and 1000 respectively. However, the obtained results of fecal coliforms for Akaki irrigation water ranges between 2,500 and 11,000,000 fecal coliform/100ml. This indicates that the river water is highly contaminated with human excreta, household and health organization wastes. When the laboratory results for total coliforms and E.colis compared to the WHO standards, the water does not fit for irrigation use. Moreover, the agronomical activities in Akaki such as cultivation of highly transmitting fresh vegetables (lettuce and carrot), hand harvest, hand cultivation of food crops, and direct contact with water as farmers wash vegetables with wastewater before bringing them to market are categorized as highly risky and expected to aggravate the health hazards. There is estimated economic loss of Addis Ababa river pollution, obviously put huge negative impact in economic loss and even affects countries GDP (Tadesse, 2011). There is no legislation that prohibits or permits the use of stream water for crop production in the study area, although campaigns try to alert people to the related risks. However, these vegetables have distributed well in the city for vegetable consumption.

Farmers have insufficient capacity to manage wastewater at field level. Hence, there is a need to conduct research focusing on exploration on the efficiency of low-cost onsite wastewater treatment, technology such as subsurface constructed wetland to minimize health risk pose by irrigation of vegetables with untreated wastewater. The findings from this research will provide baseline information for better understanding of the technology for concerned authorities and for farmers.

1.3 Objectives

1.3.1 General objective

The general objective of this study is to evaluate the treatment performance of Vertical Subsurface Flow Constructed Wetland (VSSFCW) as an alternative non-point source polluted wastewater treatment technology under Ethiopian climatic and wastewater conditions.

1.3.2 Specific objectives

The specific objectives of this research are:

1. To determine removal efficiency of constructed wetlands (VFSSCW) for selected wastewater quality parameters by taking the polluted Akaki River wastewater as a case study.
2. To determine the best process that contributes to the total removal of Faecal coliform (FC) & optimizing the FC reduction in VSSFCW using model.
3. To provide necessary information about the VSSFCW for local, regional and national governments for wider application of the technology.

1.4 Research questions

Based on the objectives of the study stated above, this research puts its roots on the following questions.

1. What is the removal efficiency of constructed wetlands (VFSSCW) for selected wastewater quality parameters for the polluted Akaki River wastewater as a case study?
2. What is the best process that contributes to the total removal of Faecal coliform (FC) & optimizing the FC reduction in VSSFCW?
3. What necessary information can be provide about the VSSFCW for local, regional and national governments for wider application of the technology?

2. LITRATURE REVIEW

2.1 Definition and rational of urban agriculture

The population living in cities is continuously increasing worldwide. In developing countries, this phenomenon is worsened by poverty, leading to tremendous problems of employment, immigration from the rural areas, transportation, and food supply. Simultaneously with the growth of cities, a new type of agriculture has emerged; namely, urban agriculture. Urbanization brings increased demand for locally grown, perishable and high-value crops, such as vegetables, which can offer farmers returns commensurate with those of urban uses. As the urban agglomeration size increases, the area devoted to peri-urban horticulture and urban agriculture may increase (De Bon et al, 2010). As vegetable production is typically labor-intensive, it contributes to employment creation (Midmore & Jansen 2003). At the same time, farming activities on the fringes of urban areas may face intense competition for water, an increasingly scarce resource in many parts of the world. The intensification of farming systems in many parts of the world, especially in proximity to urban centers, is already an observed phenomenon and poses challenges to environmental sustainability.

Urban Agriculture (UA) is the practice of growing crops near urban areas for cash earning or food producing activities. In that sense, wastewater from cities is a valuable source of irrigation water to the majority of per-urban and urban farmers. Direct use of wastewater signifies the direct use of wastewater from collection or sewage channels, whereas indirect is to mean the use of wastewater after mixed with the natural river water. Wastewater irrigation is substantially a growing worldwide practice. Globally, around 20 million hectares of land irrigated with wastewater and this figure is likely to increase during the next few decades (Hamilton et al, 2007).

Moreover, as it explained by Catherine (2000), the social and environmental benefits of UA greatly needed by cities as the world faces an unprecedented process of urbanization, the largest migration in human history. UA contributes to poverty alleviation both through a reduction of expenditures and through an increase of income, and most importantly by creating employment.

For instance, UA employs 800 million urban residents worldwide (FAO, 2001).

Despite its positive contribution to stimulate agricultural production, unwise use of wastewater for irrigation has associated adverse impacts on environment public and animal health (Sinshaw, 2011). The degradation of water quality poses serious impact on the environment, social and economic sector. The health and environmental effects of wastewater use for agriculture were well explained in Volume II of World Health Organization (WHO) Guidelines for the Safe Use of Wastewater, Excreta and Grey-water, which was published in 2006. The most common problem that wastewater use can cause in soils is salinization, which causes soil structure collapse, losing pores and interconnections that allow water and air passage. Ground water contamination due to excess use of wastewater and surface water pollution with the application of pesticide and artificial fertilizer are some of the environmental effects of wastewater use for agriculture. The primary health hazards associated with the use of wastewater, excreta and grey-water in agriculture and aquaculture, as indicated by Volume I of WHO Guidelines for the safe use of wastewater for agriculture, are excreta-related pathogens, some vector-borne diseases and certain chemicals. Heavy metals (for example, cadmium, lead, and mercury) and many organic compounds (for example, pesticides) cause health risks from chemicals. These mostly derive from industrial wastewaters and, if these are discharged to public sewers, they are present in municipal wastewaters. The well-known health effects of prolonged exposure to many of these chemicals are for example, cancers (Scheierling et al, 2010).

As Yohannes and Elias, (2017) explains both solid and liquid waste generated from previously mentioned sources contribute for the pollution of Addis Ababa Rivers. Even though Addis Ababa is the only city with sewer networks, it has a very limited sewer network coverage that accounts for 7.5% of the built-up areas. Since only parts of the older sections of the city are connected to the central sewer system, both residential and business premises use septic tanks. Currently there are efforts to control and treat the river in Addis Ababa by city administration including the recent huge river and riverside development project and few private sectors. However, these efforts are very limited and not address the whole pollution problem in the city.

In particular, the little Akaki River is more polluted than big Akaki River and has a characteristics greenish-dark color, pitch-dark sediment and a peculiar pungent odor, which is associated to the industrial and domestic waste discharged to the system. Most of the industries are established along the course of the Little Akaki River and its major tributaries. These include tanneries, breweries, wineries, distilleries, pharmaceutical and national alcohol liquor factories. According to Aaschale et al (2015) toxic trace elements in Little Akaki River exceed the permissible limits of WHO, European Community and the Ethiopian Standards of drinking water, irrigation and other uses and the river further degraded in quality as compared to the previous study. Hydrological investigation carried out in the Akaki area have shown that out of the total amount of water supplied to Addis Ababa city, 70% return as sewage of which 60% flows in to the Akaki River (Prabu, 2009). Water quality at Akaki river basin changed remarkably over the last ten years. Most of the original aquatic life in the two rivers has disappeared over the years due to high level of pollution (Mudhco, 2015).

2.2 Decentralized wastewater treatment

Wastewater has a high content of nitrogen, phosphorus, organic compounds, solids and bacteria and needs to be treated before being discharged into a water body to avoid environmental and sanitary problems (Gray, 2004). Decentralized (onsite) wastewater treatment is in many ways a more advantageous approach than conventional centralized treatment systems. Onsite treatment can be applied anywhere, in both rural and urban areas when space allows, and does not require expensive and energy-consuming transport of the wastewater in pipes. Furthermore, since there is less mixing of nutrients and other compounds it is possible to achieve a better treatment result with less energy input, and it may also be possible to reuse the water for irrigation or other purposes (Rousseau et al, 2008).

The first step in onsite wastewater treatment is usually primary treatment in a septic tank where settling of solids, flotation of grease and oils and digestion of organic matter takes place (Bounds et al, 1997). This pre-treatment is simple, low-maintenance and requires no energy to function. It may remove as much as 65% of the incoming biochemical oxygen demand (BOD) and 70% of the total suspended solids (TSS). However, in the septic tank, nutrients and pathogens are not

removed, and secondary treatment is therefore needed before the effluent can be discharged to waterways. In many countries, a septic tank is the only treatment solution, which results in contaminated wastewater being released and causing anoxia, eutrophication and spreading of potentially pathogenic bacteria and viruses.

Soil infiltration is often a preferred solution for secondary treatment due to the low cost and simplicity, but at many locations this is not possible due to soil conditions, high ground water tables or proximity to drinking water wells (Brix & Arias,2005). Over the past decades, constructed wetlands (CWs) have been gaining more and more interest as an efficient technology for secondary wastewater treatment. The advantages offered by CW systems are low construction and operation costs, low energy requirements and simple operation and maintenance (Reed et al, 1995).

2.3 Theoretical background of wetlands

The German Scientist Dr. Seidel conducted the first experiment of wastewater treatment using wetland plants in 1952. Wetlands have been referred to as a “living machine” (MacDonald, 1994) and “...one of nature’s most effective ways of cleansing polluted water” (John, 1998). They have been termed “Kidneys of the planet” because of the natural filtration processes that occur as water passes through (Wallance,1998).

Recently, the use of Constructed Wetland (CW) systems for wastewater treatment have been more popular in many developed countries such as Germany, UK, France, Denmark, Austria, USA, and Australia; however, in developing countries the application of CWs systems is still limited. The synonymous terms of “constructed” include “manmade”, “engineered”, or “artificial”, or “treatment” wetlands (Aalbers, 1998).

Ecological technologies such as constructed wetlands for wastewater treatment represent innovative and emerging solutions for environmental protection and restoration, placing them in the overall context of the need for low-cost and sustainable wastewater treatment systems in developing countries (Babatunde et al, 2008; Vymazal, 2011). CWs are engineered wastewater treatment systems that encompass a plurality of treatment modules including biological, chemical,

and physical processes similar to processes occurring in natural treatment wetlands (Kadlec & Knight, 1996; Vymazal, 2005). CWs have been successfully used to mitigate environmental pollution by removing of a wide variety of pollutants from wastewater such as organic compounds, suspended solids, pathogens, metals, and nutrients (Haberl et al, 1995; Kadlec & Wallace, 2008; Ranieri et al, 2013; Gikas et al, 2013). As well as wastewater from pharmaceutical and personal care products can treated using CW (Matamoros & Bayona, 2006; Ranieri et al., 2011; Zhang et al., 2014).

Because of high removal efficiency, low cost, simple operation, and great potential for water and nutrient reuse, CWs have become an increasingly popular option for wastewater treatment (Kadlec, 2009; Vymazal, 2007). However, to date there has been little information about the application of CWs in developing countries, and the adoption of CW technology in these countries has been surprisingly slow (Bojcevska & Tonderski, 2007). Since the application of CWs has been rapidly expanding in the past decade, emphasis of this paper is placed on the treatment performance of CWs at a macroscopic level.

All wetlands, fresh-water or salt, have many distinguishing features, the most notable of which are the presence of standing water, unique wetland soils and plants adapted to or tolerant of saturated soils (William & James, 1993; USEPA, 1993). In Ethiopian context marsh areas, swamplands, flood plains, natural and artificial ponds, volcanic crater lakes and upland bogs are treated collectively as wetland ecosystem (EIBC, 2007; Abebe & Geheb, 2003). According to Luise et al. (1999), wetlands provide a number of functions and values; wetland functions are the inherent processes occurring in wetlands; wetland values are the attributes of wetland that the society perceives as beneficial. Under appropriate circumstances wetlands can provide; water quality improvement (William, 1997), cycling of nutrients (Nichols, 1983). According to Ramsar Convention Bureau, (1997) wetland can be habitat for fish and wildlife, flood storage and the resynchronization of storm rainfall and surface runoff, Passive recreation such as bird watching and photography, active recreation (such as hunting, education and research) and aesthetics as well as landscape enhancement (Tanner & Sukias, 2003). The recognition of these wetland values and the presence of policies such as “no net loss” of wetlands in some countries have stimulated construction of wetland that have specific objectives such as the mitigation of

unavoidable wetland losses, wildlife enhancement, domestic wastewater treatment, mine drainage control, and storm water retention and control (William and James, 1993; Martha, 2003). Because of this fact, currently constructed wetlands are being used at increasing rate for treatment of wastewaters in different sources because of their consistent performance for pollutant removal.

Application of CWs systems for two decades of experience shows that the systems are generally efficient in removing total suspended solid (TSS) and Biological Oxygen Demand (BOD), meaning that the effluent concentration less than 20 mg/L (Chris, 1997). In developed countries, the aim of wastewater treatment is elimination of all pollutants like pathogen, nutrients, organic matter, and inorganic chemicals. However, in developing countries, the main goal is protection of the public health through control of pathogens to prevent transmission of waterborne disease (Abe et al, 1997). For this purpose, application of CWs in developing countries is suitable since they can be efficient in removal of BOD and pathogens but as mentioned by Christina (2005) removal of nutrients is often limited. Therefore, this paper uses the indicators of the removal of BOD, COD, pathogens (indicated by fecal coliform) and nutrients such as TN, TP to evaluate the performance of the CWs. The percentage efficiencies of the removal are known by comparing the removal of the pollutant at the outlet to the initial load of the pollutant at the inlet of the CWs.

2.4 Types of constructed wetlands and their treatment mechanisms

According to wetland hydrology, the CWs are classified into Surface Flow (SF) and Subsurface Flow (SSF). The SFCWs are similar to natural wetlands as they occupy shallow flow with low velocity above and within the media filter (substrates). In SSFCWs, wastewater flows horizontally or vertically through the substrates. Hence, the SSF systems are classified into Vertical Flow (VF) (also called Down Flow Treatment Wetland) and Horizontal Flow (HF). The combination of VF and HF constructed wetland is known as Hybrid CW (Addis Ababa city municipality, 2004). The CW basin commonly contains a combination of substrates and macrophyte plants. The process of purification occur during wastewater do contact with the

surfaces of the substrates and the plant roots.

Lienard *et al.*, (2001) termed the vertical flow treatment wetlands as Down Flow Treatment Wetlands (DF TW) and they are very similar in design to intermittent sand filters which are widely used throughout the USA, Australia and New Zealand for decentralized Wastewater treatment, except that DF TWs is planted with wetland vegetation. DF TWs are used in many European countries particularly for achieving secondary treatment of primary settled sewage. Due to their higher oxygen transfer rates, DF TWs are becoming more common where discharge regulations require removal of ammonium. They are also a common design choice for effluents with a high carbonaceous or nitrogenous oxygen demand, such as landfill leachates and agricultural wastewaters.

2.4.1 Surface Flow Wetland

Surface flow constructed wetland(SFCW) systems most resemble natural wetlands both in the way they look and the way they provide treatment and can be used to treat wastewater from individual and community sources, but surface flow wetlands are usually more economical for treating large volumes of wastewater (Sinclair, 2000). The General characteristic of SF wetland is that water level is above the ground surface; vegetation rooted and emerges above the water surface; water flow is primarily above ground. The surface flow (SF) wetland typically consists of a shallow basin, soil or other medium to support the roots of plants and a water control structure that maintains a shallow depth of water (Luise et al, 1999).

As soon as wastewater enters to surface flow wetland cell, natural processes immediately begin to break down and remove the waste materials in the water (Renee, 2001; Kaseva, 2003). Before the wastewater has moved very far in the wetland small-suspended waste materials are physically strained out by submerged plants, plant stems, and plant litter in the wetland (Hammer, 1992). Floating plants, such as water lilies and emergent plants, such as cattails, shade the water surface and control algal growth (Sinclair, 2000). The roots, stems, leaves, and litter of wetland plants also provide a multitude of small surfaces where wastes can become trapped and waste-consuming bacterial can attach themselves to the plant (USEPA, 1993;

Sinclair, 2000). These bacteria provide the majority of wastewater treatment. Wind, rain, wastewater and anything else that agitates the water surface can add oxygen to the system. This helps the aerobic bacteria thrive in wetlands near the surface wherever oxygen is present, in addition to this, anaerobic bacteria thrive in the wetland where there is little or no oxygen (USEPA, 1993). When bacteria consume waste particles in the water, they convert them into other substances such as methane, ammonium, sulfate, orthophosphate, carbon dioxide and new cellular material. Some of these substances are used as food by plants and other by bacteria (Christina, 2005).

For any of the processes in wetlands to work, the wastewater must remain in the system long enough for treatment to occur naturally. The hydraulic residence time for wastewater in SF systems based on wastewater strength, the level of desired treatment and climatic factors (William & James, 1993; Sinclair, 2000). The advantages of SF wetlands over SSF wetlands are that their construction, operation, and maintenance are straightforward. The main disadvantage of SF is its requirement of a larger land area than other systems (Luise et al, 1999).

2.4.2 Subsurface flow wetland

The Subsurface flow (SSF) wetland, which is originated in Europe over 40 years ago (Kyambadde, 2005), consists of a sealed basin or channel with a porous substrate of rock or gravel media and a barrier to prevent seepage. The media also support the root structure of the emergent plants. The design of this system assumes that the water level in the bed will remain below the top of the rock or gravel media (USEPA, 1993; Luise *et al.*, 1999; Martha, 2003).

The treatment processes in SSF wetland system is more efficient than in the SF wetland system; because the media provides a greater number of small surfaces, pores and crevices, where treatment can occur. Waste consuming bacterial attach themselves to the various surfaces, and waste materials in the water become trapped in the pores and crevices on the media and in the spaces between media. Chemical treatments also takes place as certain waste particles contact and react with the media (USEPA, 1993). Consequently, the treatment responses may be faster in SSF type, which is smaller in area than a SF system designed for the same wastewater

conditions (Wallace, 1998; USEPA, 1993). The biological treatment in SSF wetlands is mostly anaerobic, because the layers of media and soil remain saturated and unexposed to the atmosphere (Sinclair, 2000). However, wetland plants are able to grow extensive roots even in these anaerobic conditions.

The SSF type of CW is thought to have several advantages over the SF type since the water surface is maintained below the media surface with little risk of odors, and insect vectors and elimination of the risk of public Contact with the partially treated wastewater (Vymazal, 2002). In contrast, the water surface in natural marshes and free water surface (FWS) constructed wetlands are exposed to the atmosphere with the attendant risk of mosquitoes and public access.

2.4.3 Differences between wetland types

One difference between (Horizontal Flow Constructed wetland) HFCWs and (Vertical Flow Constructed wetland) VFCWs is their oxygen availability (Randerson, 2006). In HFCWs the wastewater is pumped into the wetland from the storage tank, right into the gravel medium of the bed. Therefore, it does not have contact with the outside air and thus does not take oxygen into the bed. Oxygen reaches the wetland through diffusion of oxygen from the surface to the water in between the gravel in the bed and through the roots of the plants. By this limitation of oxygen in HFCWs, some pollutant removal mechanisms are limited.

In VF wetlands, the water is distributed over the surface area of the wetland by pipes. During the infiltration to the bottom, the water is able to dissolve oxygen that is present in the pores of the surface layers of the wetland (Randerson, 2006).

Another difference between HFCWs and VFCWs is their hydraulic conductivity (Kadlec, 2000). HFCWs, an adequate hydraulic conductivity is harder to achieve than in VFCWs. This is because in HSSFCWs the hydraulic conductivity depends on the slope of the bed, whereas in VFCWs, the water is moved through the bed by gravity force from surface to bottom. In addition, the hydraulic conductivity depends on the type of material used as medium in the bed. Sand is less suitable for HFCWs, because the horizontally formed pores collapse more easily than the vertically formed pores in VFCWs. To avoid this problem, the medium of the HSSFW

has to have a sand-clay mix to support the growth of plants and gravel to support the formation of pores (Brix & Arias, 2005).

Surface Flow Wetland (SFW) significantly distinguishes itself from the HFCW and VFCW due to the absence of gravel or sand medium that serves as a filter in the HFCW and the VFCW. In SFW with emergent plants, soil on the bottom of the bed is only present to support the roots of the plants (Peter & Alette, 2014). The operational regime of pulse loading, followed by rest periods, allows VF systems to operate in a mode of unsaturated flow, which allows the introduction of air (and oxygen) into the VF bed (Cooper, 1999).

2.5 Design of a subsurface flow constructed wetland (SSFCW)

2.5.1 Pilot plants

Pilot plants or small-scale plants are commonly used to demonstrate new technology or for application of a specific technology at a site. Until the full performance of pilot plants can be assured, installation at water or wastewater treatment facilities may represent a threat to public health or environmental quality (EPB, 2012).

2.6 Model review

2.6.1 Black-box model category

It is realized that most models used in CWs were based on input/output data (Rousseau et al., 2004). In addition, the treatment processes in wetland were considered as a black-box, as illustrated in Figure 1.

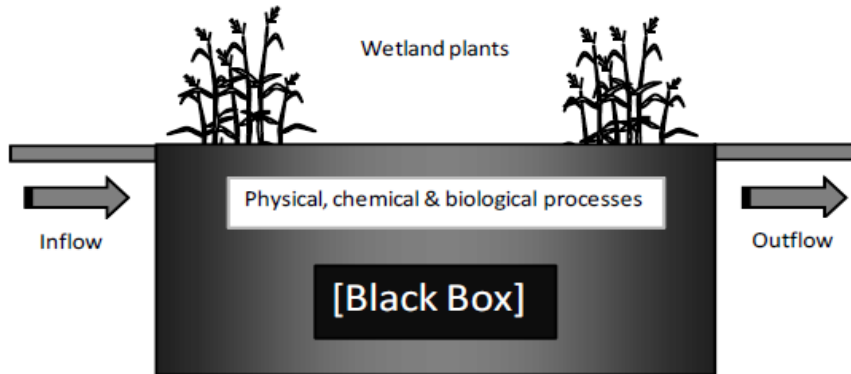


Figure 1. Schematic illustration of wetland process

Even if CWs are widely used and studied, they are often described as “black-boxes” where the interactions between soil vegetation, water and microorganisms are not well known (Toscano et al., 2009). This perplexing situation leads to numerical models with different complexities to better understand the process in CWs.

2.6.2 Rules of thumb

From an engineering point of view, rules of thumb are the fastest but also the roughest design methods. As an example, some of these rules for horizontal SSF constructed treatment wetlands are described by Wood, (1995) ; Kadlec, and Knight, (1996). Since they are based on observations from a wide range of systems, climatic conditions and wastewater types, these rules of thumb show a large variation.

2.6.3 Regression equations

Considering the fact that the majority of the investigations on treatment wetlands have mainly been focused on input–output (I/O) data rather than on internal processes data, regression equations seem to be a useful tool in interpreting and applying these I/O data. However, these black box ‘models’ lump a complex system like a constructed treatment wetland into only two or three parameters, which is clearly an over simplification. Important factors such as climate, bed material, bed design (length, width, and depth), etc. are neglected, leading to a wide variety of

regression equations and thus a large uncertainty in the design. For instance at a literature overview of regression equations for BOD, COD, TSS, TN and TP is presented by Brix.H, (1994) implies that for a constant BOD influent concentration, the same effluent concentration is predicted for a Hydraulic loading Rate (HLR) of 0.8 as well as 22cmday^{-1} , which suggests that the HLR is a non-limiting factor within certain boundaries. Only a limited number of regression equations rely on both influent concentration and HLR as inputs to predict the effluent concentration. Consequently, only those regression equations can be used to predict the maximum allowable HLR based on a given influent concentration and a given effluent standard.

2.6.4 Mechanistic, compartmental models

Only recently, Wynn & Liehr (2001) have presented a mechanistic, compartmental simulation model of a SSF constructed treatment wetland. The model consists of six interlinked sub models, representing the carbon and nitrogen cycles, the water and oxygen balances, and the growth, decay and metabolism of heterotrophic and autotrophic bacteria. Removal of phosphorus and suspended solids is not modeled since they mainly depend on physical and not on biological processes. Hydraulic behavior is modeled via a tanks in-series approach to mimic the mixing regime, and via the Darcy equation to imitate flow in a porous medium. The required model inputs are air temperature, day length, precipitation, flow rate and the concentrations of BOD, $\text{NH}_4 - \text{N}$, $\text{NO}_3 - \text{N}$ organic N and dissolved oxygen. The model output consists of flow rate and the same five concentrations as for the input. The dynamics of the 15 state variables are modeled via 15 ordinary differential equations that contain 42 parameters related to physical, microbiological and biological processes. On the one hand, this complexity of the model enables to better summarize the processes that occur within constructed wetlands as well as to demonstrate interactions between certain components. On the other hand, it requires estimation of 15 initial conditions for the state variables and knowledge about or estimation of 42 parameters, which is not an obvious task. The authors calibrated the model only approximately by adjusting the parameters to a certain extent to make the model output corresponding with the available site data. This procedure however yielded values for several microbial parameters that were one or more orders of magnitude lower than those typically mentioned in the literature. Due to the complexity of the model, it is very well possible that certain parameters compensate for each

other, thus causing insensitivity to parameter changes please refer (Dochain & Vanrolleghem, 2001). However, it would be more reasonable to assume that certain important phenomena were not included in the model, although they are influencing microbial reactions.

2.6.5 Non-mechanistic models

Inside the non-mechanistic models, first-order reaction in an ideal plug flow reactor (K-C), that can be modified (K-C*) including a background concentration (C*) that represent the remaining non-biodegradable pollutant (Kadlec & Knight, 1996).

2.6.5.1 First-order models

The state-of-the-art in constructed treatment wetlands' modeling consists of first-order equations (Kadlec RH., 1997). According to Diederiket et al (2003) this model is used in case of constant conditions (e.g. influent, flow and concentrations) and an ideal plug-flow behavior predict an exponential profile between inlet and outlet.

$$\left(\frac{C_{out}-C^*}{C_{in}-C^*}\right)=e^{(-K_A/q)} \quad \text{Equation 2.1}$$

$$q = Q/A \quad \text{Equation 2.2}$$

Where $C_{in} = C(t = 0)$ and $C_{out} = C(t = \tau)$ initial conditions, C_{in} inflow concentration (mg/L), C_{out} outflow concentration (mg/L), C^* = background concentration (mg/L), A = required area (h), Q = water flow rate, (m^3/d), k_A is the areal decomposition constant (m/yr.), q =hydraulic loading rate (m/yr.),

The influence of temperature is commonly modeled via Arrhenius equation and for the values at 20°C please see table 1.

$$K_{A,T}=K_{A,20}\theta^{(T-20)} \quad \text{Equation 2.3}$$

Where: K_{AT} =first order Ariel rate constant at temperature $t^\circ C$, K_{A20} = first order ariel rate

constant at 20°C, θ = temperature correction factor, T= Wetland water temperature °C.

Table 1.Kadlec and Knight K- C^*Model Design Parameter

Parameter	$K_{A,20}$	θ	C*(mg/L)
BOD	34	1.00	3.5+0.053Ci
TSS	1000	1.00	7.8 + 0.063Ci
Organic-N	17	1.05	1.5
TN	22	1.05	1.5
TP	12	1.00	0.02
Fecal coli	75	1.00	300cfu/100ML

Adapted from Treatment wetlands (P.217), by R.H. Kadlec and R.L. Knight, 1996, Boca Raton, FL: CRC press.

Area of pilot VSSF CW is calculated using Equation 1.4 after rearranging the First-order model equations.

$$A = \left(\frac{0.0365 \cdot Q}{K_A} \right) \times \ln \left(\frac{C_{in} - C^*}{C_{out} - C^*} \right) \quad \text{Equation 2.4}$$

According to (Mena et al, 2008) First-order model is valid for both types of SSF CW if it is used in the areal form.

Assumptions needed for the development of the model are as follows; the wetland is in a stationary state, so there are no adaptation trends. The wetland has spatially invariant averaged flow, or precipitation equals evapotranspiration, there is no infiltration, there are no inputs into the wetland via atmospheric deposition. The wetland is rectangular and exhibits plug flow conditions (i.e., no back mixing or bypassing), there is no variation in the cross-flow direction (Garcia, 2015).

As Kadlec (2000) pointed out the inadequacies of first order model are due to the variability caused by unpredictable events such as fluctuation in input flows and concentration in internal

storages, as well as by weather, animal activity and other ecosystem factors. However, the first-order model is still considered as an appropriate design equation for pollutant removal in Constructed Wetlands (Kadlec & Wallace, 2009).

2.6.6 Variable-order or Monod-type models

Mitchell and Mcnevin (2001) identified another physical impossibility of the first-order model, namely the fact that the removal rates continue to increase with increasing loading rates. However, in most cases, a maximum allowable loading rate has been demonstrated. Therefore, Mitchell & Mcnevin (2001) advocate the use of a Monod-type design model, which represents first-order rate reactions for relatively low concentrations but zero-order rate reactions for high concentrations with the assumption of plug flow. One of the interesting features of this model is an alternative explanation of background concentrations (C^*). Indeed, if concentrations drop to near zero, the Monod equation predicts a very low reaction rate, which may prevent total decomposition of the pollutant within the given hydraulic retention time. The authors did not try to assess parameter values, but used a graphical representation of loading and removal rates from the North-American treatment database to extract some design parameters (Knight et al, 1993). UK systems (Green & Upton, 1992) show most actual loading rates well below these maximum recommended levels. Several exceptions are however mentioned where, despite significantly higher loading rates, effluent concentrations are still of acceptable quality.

2.6.7 Process-based models

Process-based models allow the increased understanding of the processes occurred in the “black-box” CWs (Langergraber, 2007). These models can provide insight into the “black-box” and gives generous information, which helps highly for the design purpose.

Although the physical and biological processes that drive wetland systems are complex, many mathematical models have been developed to simulate nutrient removal in wetlands. Many of these models were developed by accounting for hydrologic conditions and nutrient dynamics. A mathematical model was developed from studies of lowland rice fields and can be used to assess the extent of absorption from the rhizosphere by wetland plants growing in flooded soil,

incorporating important plant and soil processes (Kirk & Kronzucker, 2005). McBride and Tanner (1999) developed a mathematical model to simulate patterns of nitrogen removal that were observed in experimental studies of constructed wetland.

2.6.8 FITOVERT model

Relatively few numerical models specifically developed to simulate CWs have been reported (Brovelli et al, 2009). Most of the currently available models can simulate HSSFCWs (Giraldi et al, 2010) but only few models can simulate VSSFCWs (Langergraber & Simunek, 2005). To bridge the gap in VSSFCWs, Giraldi et al. (2010) developed a mathematical model, called FITOVERT. It can simulate the hydraulic behavior of VSSFCWs in both saturated and unsaturated conditions. Biodegradable Organic matter (OM) and Nitrogen compounds in FITOVERT model was developed by using activated sludge model one (ASM 1) (Henze et al., 2000). On the other hand, FITOVERT can also handle the porosity reduction due to bacteria growth and accumulate of particulate components, so that the clogging process is also simulated as an effect of pore size reduction on the hydraulic conductivity of the simulated system.

2.6.9 Constructed Wetland two Dimensional (CW2D) model

The first implemented HYDRUS-2D was used as a starting point for the CW2D implementation. However, the software is now called HYDRUS (Simunek et al., 2006, See <http://www.pc-progress.com>). Langergraber and Simunek (2005) developed the multi-component reactive transport model CW2D for sub-surface flow CWs. as an extension of HYDRUS-2D variably saturated flow and solute transport package. Biochemical transformations in CW2D are based on the ASM (Henze et al. 2000). The main drawback of CW2D is that up to now, only dissolved substances are considered and it is necessary to consider particulate wastewater constituents for the realistic model (Langergraber and Simunek, 2005). Langergraber (2003) used CW2D model which consists of different layers (main, intermediate and drainage) filled with various size of the gravel planted with *Arundo donax* (giant reed) to focus mainly on the hydraulic behavior of the CWs. Results reveals that the reactive transport simulations with CW2D fit the measured data well for the pilot scale CWs.

2.6.10 STELLA

Software STELLA is a graphical programming language especially for system dynamics study. To model and better understand the non-linear dynamic systems in CWs many researchers used STELLA graphical programming language such as Wang & Mitsch (2000), Ahn & Mitsch (2002) and Ouyang et al. (2010). Pimpan & Jindal (2009) explained the adsorption, desorption and plant uptake in the laboratory scale FWSF-CWs planted with bulrush (*Cyperus*, *Corymbosus* & *Rottb*) using the STELLA software. The simulated and measured average cadmium (Cd) removal efficiencies were in the range of 61.7- 99.6 % and 74.6 - 96.5 %, respectively. Since the measured and simulated values are in good agreement, it has been recommended to use the developed mathematical model for the Cd removal. Mayo & Bigambo (2005) studied the process of Nitrogen (N) transformation in HSSCWs. It has been found that the sedimentation and the regeneration mechanisms accounted for $0.872 \text{ g/ m}^2 \text{ /d}$ and $0.752 \text{ g/ m}^2 \text{ /d}$ of N transformation respectively. Significant transformations were also observed through denitrification and nitrification which were responsible for $0.436 \text{ g/ m}^2 \text{ /d}$ and $0.425 \text{ g/ m}^2 \text{ /d}$ of transformed N respectively. However, N removal through plant uptake requires plant harvesting from the wetlands.

2.6.11 PHWAT Software

Brovelli et al. (2009) presented a modular modeling tool suitable for simulating the clogging process in 1, 2 and 3D. A new clogging module was implemented for the numerical model which evolved from PHT3D. PHWAT is a computer code for 3D reactive transport in variable-density saturated flow. This numerical model is able to simulate the effect of biomass growth on the hydraulic properties of saturated porous media, i.e. bio clogging. The model is developed at the macro-scale, and includes the effect of flow-induced shear stress on biofilms. This model has greater flexibility because of an arbitrary reaction network and the multiple components can induce pore clogging. The simulation results demonstrated that the rate and patterns of bio clogging development are sensitive to the initial biomass distribution.

2.6.12 2D mechanistic model

Ojeda et al., (2008) used a two-dimensional (2D) mechanistic mathematical model in order to evaluate the relative contribution of different microbial reactions to organic matter removal (in terms of COD) in HSSF-CWs that treated urban wastewater. The model is based on the code Retraso Code Bright, which has been modified to include the main microbial processes related to organic matter and nitrogen transformations in the wetlands. In their study, they also evaluate how changes in the organic loading rate affect both organic matter removal efficiency and the relative importance of the microbial reactions.

2.6.13 Constructed Wetland Model No.1 (CWM1)

Langergraber et al., (2009) presented a general bio kinetic model to describe biochemical transformation, degradation processes for organic matter and nitrogen in subsurface flow CWs. CWM1 considers the bio kinetic processes in HF and VF CWs, and the main objective is to simulate the effluent concentration. They suggested to include other processes including porous media hydrodynamics, the influence of plants, the transport of particles or suspended matter to describe clogging processes, adsorption and desorption processes and physical re-aeration must be considered for the formulation of a full model for constructed wetlands. It is believed that CWM1 such as the IWA ASMs will become a widely accepted model formulation for biochemical transformation and degradation processes in subsurface flow CWs and will be implemented in many simulation platforms.

2.6.14 Winding up and approval among the models

It is generally accepted that the CWs may enable the effective, economical and ecological treatment of agricultural, industrial and municipal wastewater. Among the black box models the first-order model is still widely recognized for the design of CWs (Kadlec and Wallace, 2009). The rules of thumb seemed to be the more conservative design models. Since these are easily applicable, designers could be tempted to stick to those models (Wood, 1995). However, they may be guaranteeing good quality effluent, but they will likely be counteracted by economic constraints. conservative designs tend to increase the investment costs. The mechanistic model of

Wynn & Liehr (2001) on the other hand did not offer real help for design purposes due to some assumptions and empirical relations that are not physically based and thus corrupt model output. Hereafter, none of these “black-box” models explains the internal process mechanisms of Constructed Wetland.

At present, the state-of-the-art k- C* model seems to be the best available design tool if the designer makes sure that all the assumptions are felled. Accordingly In this research for the designing purpose of the pilot scale VSSFCW the k- C* model was selected among the other black box models. And from the above discussed process based models the most used in academic and research area; STELLA software was selected to determine the best process that contributes to total removal of FC & optimizing the FC reduction in VSSFCW. The choice of software was based on its applicability in the modeling context especially of the water and wastewater quality aspects, user friendliness and availability.

2.7 Definition of the problem

The problem to be solved has to be clearly defined. All the forcing functions selected are used to evaluate the extent to which they influence the state variables and the process of fecal coliform removal (Jorgensen & Bendoricchio, 2001). In this study the problem was to quantify each process that contributed on the total removal of FC hence to see the possibility on optimize the FC reduction in VSSFCW. The state variables in this study were FC, BOD5 and NH_3 while the forcing functions were DO, temperature, pH and flow rates. The state variables and forcing functions in this study were linked by the sub model.

2.8 Sub model

Sub models are mathematical equations that presented in the removal mechanisms, which include inflow, outflow, sedimentation, filtration, die-off and growth processes.

This is the stage where different equations to describe processes are considered. Depending on the complexity of the processes, the equations can be of first order kinetics, Monod equations etc. Conservation of mass equations are developed for interactive physical and biological species

reacting to environmental factors that affect the mechanisms (Jorgensen & Bendoricchio, 2001).

2.9 Verification, calibration and validation

Verification is to some extent a subjective assessment of the behavior of the model. Largely, the verification will go on during the use of the model before the calibration phase (Jorgensen & Bendoricchio, 2001). One of the techniques on verification is to disturb the input variables and study the behavior of the output. If model is correct and the parameters are properly defined then the constants will remain constant regardless of the level of the input (Mashingia, 2006).

Calibration is an attempt to find the best accordance between computed and observed data by variation of some selected parameters. It may be carried out by trial and error or by use of software developed to find the parameters giving the best fit between observed and computed values (Jorgensen & Bendoricchio, 2001).

Validation consists of an objective test of how well the model fit the data. This is done by running the model with a new set of data with physical parameters and forcing function to relate new condition. The model is revised each time until it is verifiably and consistently accurate. In contrast, the kinetic coefficients are kept fixed at the values derived the original calibration. When the new model is validated, it becomes an effective tool for the range of conditions defined by the original calibrated and validation data set (Jorgensen & Bendoricchio, 2001).

2.10 Description of design parameters that affect removal mechanisms

2.10.1 Hydraulic retention time

Hydraulic retention time (HRT) is an important design parameter affecting the effluent water quality (Rai et al., 2013). The HRT is the average residence time of the water in the wetland. As described by Rai (2013) HRT has to be long enough to allow the removal processes to take place. Settlement of suspended solids, adsorption and biodegradation/transformation of suspended and dissolved organic compounds. If that does not happen, the pollutants will be washed out with the effluent. Rai et al., (2013) further showed that a HRT of 12 hours leads to a

decrease in NO_3-N , NH_4-N , PO_4-P , BOD and TSS with 49.2%, 33.5%, 33.6%, 6.6% and 15.3% respectively. It also resulted in an increase in DO. On the other hand, Li et al., (2014) reports an average HRT of 1-2 days for VFCWs. However, Miriam *et al.* (2002) showed that HRT could increase even more approximating the minimum HRT to remove 50% of the bio-reactive phosphate to be 7 days. For example Nyakang'o and van Bruggen, (1999) obtained 88% removal efficiency for orthophosphate This might be due to high HRT obtained by using serious cells to maximize the contact area and time (Birhanu, 2007).

Crites and Tchobanoglous,(1997) concluded that Most subsurface flow wetlands are designed so that wastewater travels through the length of the cell one time to receive treatment. Typical retention times range from two to five days for BOD5 removal and seven to fourteen days for nitrogen and phosphorus removal. Due to this, SSF wetlands have most frequently been used to reduce BOD5 from domestic wastewaters (Godfrey *et al.*, 1985; Rechar, 1998).

Many studies have shown that the effect of hydrologic and hydraulic conditions show inconsistent results. Hydrologic and hydraulic conditions in a wetland can influence the efficiency of processes that remove nutrients from water (Christos & Vassilios, 2007). For Example Schulz et al., (2003) found that Hydraulic residence time was negatively correlated with TN and phosphorus removal in constructed subsurface flow wetlands. In addition to this NH_4^+ and total Kjeldahl nitrogen (TKN)¹ concentrations with in a wetland decreased exponentially with increased residence time (Huang et al., 2000).

2.10.2 Hydraulic conductivity

Hydraulic conductivity is a parameter that indicates the ease with which water can flow through the gravel bed (with unit m/d). It depends on the available volume of the wetland. The available volume is a sum of all the empty spaces and the pores in the bed. In other terms: the hydraulic conductivity states the relation between friction and water velocity (Kadlec & Knight, 1996). In order to be able to calculate the hydraulic conductivity, we must make a distinction between two types of flow regime, laminar and turbulent flow. In a Subsurface Flow Constructed Wetland

¹TKN (Total Kjeldahl Nitrogen) is the total concentration of organic nitrogen and ammonia Knight *et al.* (2000).

(SSFCW), the size of the granular media is determining for the type of flow regime (Kadlec & Knight, 1996). The hydraulic conductivity is an important design parameter, because it determines how well the wetland functions as a filter or how clogged the wetland is (a constraint to efficient filtering from suspended solids). For wastewaters with a high TSS concentration, the hydraulic conductivity must be higher compared to wastewaters with a lower TSS concentration, to allow passing through of the water and prevent clogging (Zhang, 2012). The wastewater information sheet, (2010) further describes that depending on the kind of plants used, hydraulic conductivity increases with time as old rhizome channels remain open after the rhizomes decayed there by creating a series of pores through the bed. They can develop in any significant quantity after three to five years.

2.10.3 Sludge retention time

Sludge retention time (SRT)² depends on the maximum growth rate of microorganisms (Clara et al., 2005). The SRT is therefore important design parameters for all wastewater treatment plants (WWTPs) that perform on biodegradation/transformation of pollutants. High SRTs support the variety of removal pathways that can take place and therefore supports the removal of a broader variety of pollutants with accordingly lower effluent concentrations of pollutants.

In conventional WWTPs such as activated sludge systems, excess sludge is being removed to maintain a constant available volume for the wastewater. In CWs, on the other hand, the sludge is not being removed on purpose. The removal of sludge in CWs is only dependent on the decay rate of the microorganisms. This results in SRT much higher than in activated sludge systems, membrane bioreactors and other WWTPs depending on sludge retention (Peter & Alette, 2014).

2.10.4 Substrates

Substrates used for CWs include soil, sand, gravel, crushed rock, other natural materials, as well as organic materials. When available, volcanic rock is the best medium but other materials such as limestone, river rocks, recycled concrete and recycled crushed glass to desired diameter are also being used. The gravel is the growth medium for microorganisms, works as a sieve and

² Defines the time that the microorganisms (in the form of sludge) are retained in the system (Clara et al., 2005).

determines hydraulic residence time they act for growing medium of macropyhte plants, support biochemical and chemical transformation.

As the research done by Li et al., (2014) indicates Important characteristics of the granular medium in the wetland are Size of granular medium which represents hydraulic conductivity and Mineral content that is physiochemical removal mechanisms. Systems with sandy substrate had low conductivity and, therefore, needed to be larger to generate a retention capacity effective at removing nutrients, which requires more land surface for construction and operation (USEPA, 1993). On the contrary Beds with smaller gravel size showed better effluent quality and higher removal efficiency according to Garcia et al., (2005). This can be explained due to the higher available area on the smaller gravel, which allows for more microorganisms to attach onto it. So during construction it must be chosen which one is the most important constraint for the target use of the effluent quality; either the hydraulic conductivity or the nutrient removal.

According to (Rob, 2010) it is crucial that the gravel or sand be clean, washed and without impurities. If porosity of the original gravel declines (which can occur over a period of 5 years to 15years later, depending on the nature of the wastewater), new gravel can be substituted or the original gravel removed and cleaned. The plants can then be transplanted back in and the system can continue effective.

2.10.5 Background concentrations

Wetlands are most often configured as flow-through systems, with water entering at one end and exiting at the opposite end. Along the flow path, there is typically an exponential decrease in concentration, from the inlet value to a lower exit value (Kadlec & Knight, 1996). If the wetland is large compared to the added water flow and chemical load, then the concentration displays a plateau towards the outlet of the system, along which further reductions do not take place. The concentration along the plateau is the treatment wetland, background concentration. Kadlec and Knight (1996) explain several potential reasons for a background concentration. First, there may be some portion of the incoming chemical that is resistant to storage or conversion in the wetland environment. This is particularly possible when the concentration

measures a lumped set of species, one or more of which may be resistant to degradation in the wetland. A second reason for a non-zero background concentration is the association of the chemical with particulates. For instance, in wetlands with more open water surface, the presence of phytoplankton in the water column is enhanced because of high light levels. The presence of these organisms implies an accompanying suspended particulate phosphorus concentration. A third reason for a non-zero background concentration is a set of wetland processes that provide inputs distributed across the entire areal extent of the system. Groundwater discharge and rainfall may bring a specific compound into all portions of a wetland. The chemical may be utilized in the biogeochemical cycle, which is also distributed across the entire wetland area. That same cycle can produce return of the substance to the water column, usually by the processes of decomposition and leaching (Kadlec RH., 1997). The determination of the wetland backgrounds done by parameter estimation from long-term data sets, or by selecting the lowest observed concentration in the dataset. Values for SSF systems are typically lower than for FWS, because the gravel matrix isolates the flowing water from some of the background-generating processes. For instance, pathogen background is sometimes associated with wildlife activity, which is minimal or non-existent in the subsurface water. Additionally, the biogeochemical cycle returns some portions of some constituents to above-gravel locations. In general, background concentration differences are not sufficient to influence the decision on wetland type (Kadlec & Knight, 1996).

2.10.6 Aspect ratio (L: W)

The configuration of wetland cell is important in basin design because of its impact on flow resistance and hydraulic circuiting. There is much information in the literature on the effects of aspect ratio (length-to-width, L: W) on the performance of ponds and wetlands for pollutant removal. Kadlec and Knight (1996) suggest the aspect ratio L: W should be greater than 2:1 to ensure plug flow conditions. However, a very high aspect ratio will increase the effective detention time and thus may lead to overflow problems due to gradual accumulation of vegetation litter (Kadlec & Wallace, 2009). Hence, commonly used aspect ratios are between 2:1 and 5:1. Persson, Somes, and Wong (1999) present a relationship between hydraulic efficiency index and the aspect ratio. Their study shows that the aspect ratio should be greater than 1.88 but less than 5

so that the efficiency performance will be “satisfactory”.

Zhang (2012) also adds his finding on the effect of aspect ratio on the hydraulic efficiency, and concludes that the hydraulic efficiency increases with the aspect ratio. However, the hydraulic efficiency of the wetlands significantly improves when the aspect ratio is less than five. On the other hand the Barcelona studies of García et al. (2004) found no effect of aspect ratio on performance, within the range $1:1 < L:W < 2.5:1$; but that range of aspect ratio is quite small.

The aspect ratio for this research was assumed to be around 3:1 in order to grant the plug flow and reasonable high detention time, for that reason this value is selected between commonly used aspect ratios between 2:1 and 5:1.

2.10.7 Depth and water drainage

As explained by Chinniah et al., (2000), the active treatment depth is 0.1 to 0.6 m with an average permanent wetland water depth at 0.3 m. Increasing the water depth can increase the effluent COD concentration. Garcia et al (2005) explains that shallow beds allow energetically more favorable biochemical reactions to take place for the removal of COD. Also, with increasing depth, the DO concentration decreases, impeding the oxygen depending reactions. In other words, water depth influenced the redox conditions of the bed. Like Garcia et al. (2005), Matamoros and Bayona (2006) also state that the water depth influences the removal efficiency. They explain it with the higher redox potential that reached in shallower beds. Also it is important to ensure that drainage is adequate around a CW so that runoff rainwater and soil do not wash into the system. Systems should be built with a protective beam higher than surrounding ground level and one must check occasionally to make sure soil has not built up around the system, which will allow rain runoff to enter (Rob, 2010).

2.10.8 The role of plants

In the first place, the roots of the plants are very useful for oxygen supply and for the uptake of nutrients (Rob, 2010). According to Brix (1994), wetland plant roots contribute oxygen to the cells, which allow some aerobic treatment to take place in the root zone, which stimulates organic

matter oxidation and the growth of nitrifying bacteria. This is because vascular wetland plants are equipped with aerenchyma or aerenchymous tissues containing a network of tiny hollow tubes that traverse the length of the plant allowing gases to move from the above water part of the plant to the roots and rhizomes, and vice versa (Richard, 1998). However according to Rob (2010) Plants play the role of aerators although transfer from plants is minimal (about 0.02 g/m^2 per day Oxygen).

Other than the aesthetics of the wastewater treatment plant, the root system increases the surface available to bacterial colonization, providing substrate to the microorganisms. Their roots support symbiotic bacteria and fungi, resulting in a more diverse microbial environment. Allen et al, (2002) Further explains that Plants are responsible for providing and maintaining hydraulic pathways in the substrate as well as up taking nutrients such as Nitrogen or Phosphorus (although to a limited extent). Despite the disadvantages, plants aid in the removal of pollutants in many ways and their presence is very important. Rai (2013) showed a correlation between the water quality and the development of plants in the wetland. Reduction of BOD, TSS, TDS, nitrogen and phosphorus increased the growth of the plants. Until they are full-grown, their maximum removal capacity is not reached. In addition, they are more vulnerable to toxins during this time (Rai et al., 2013). Light attenuation by the plants can also be an advantage, because this prevents algae to grow according to (Brix H. , 1997).

The higher efficiency of the planted beds in removing COD and BOD compared to the unplanted beds indicates that plants were able to oxygenate the beds to a level that supports the aerobic degradation of the organic load of the sewage. In addition, the vegetation provides a substrate (roots, stems, and leaves) upon which microorganisms can grow as they break down organic molecules (Sara et al, 2016).

A recent study of nitrogen uptake in the rhizosphere concluded that nitrate (NO_3^-) uptake by wetland plants may be far more important than previously thought. According to Kirk and Kronzucker, (2005) model calculations showed that substantial quantities of NO_3^- can be produced in the rhizosphere of wetland plants through nitrification and taken up by the roots under field conditions and those rates of NO_3^- uptake can be comparable to those of NO_4^+ . In

addition, the model showed that rates of denitrification and subsequent loss of nitrogen from the soil remain small even where NO_3^- production and uptake are considerable.

As it was studied by Godfrey et al., (1985) and Richard, (1998) Plants also take-up and store some of the metals and nutrients in the wastewater.

Many studies have shown that different species of plants perform better than others at nutrient removal from wastewater. As a result, Careful consideration should be given to the choice of plant species used for nutrient removal systems. Because as studied by Mitsch and Gosselink, (2000), While many species can be desirable and effective for nutrient removal in some regions, those same plants can be undesirable in other regions and can often be highly invasive, spreading to and even causing problems in nearby aquatic systems. In addition to this, the study conducted by Jing et al. (2001) makes as sure that the Removal efficiencies for organics, NO_3 and orthophosphates were influenced by the health and growth rate of macrophytes. Macrophyte plants are commonly found in natural wetlands or riverbanks. In tropical developing countries, the macrophyte plants grow very well. Emerged plants such as Cattail, Bullrush, and Common Reed were recommended to be applied in SSFCWs (Reed et al, 1995).

In warm climates of South America, Africa, and Asia, suggested the types of Macrophyte plants for SSFCWs that includes decorative plants (Papyrus sedges or *Cyperus papyrus*, Bamboo plants, Heliconia, Canna lily, Calla lily), very robust plants (Umbrella sedge or *Cyperus albostraitus* and *Cyperus alternifolius*, Dwarf papyrus or *Cyperus haspens*), and Broad-leaved cattail (*Typha latifolia*), as well as Species of grass native to the tropical grasslands (Napier grass or Elephant grass).

2.10.8.1 Description of macrophytes of the study area

Jehovah's Witnesses Branch Office constructed wetland is covered with three different plant species, identified in the local areas, namely *Cyperus papyrus*, *Cyperus alternifolia* and *Phoenix canariensis*, which are among the typical characteristic species of wetland ecosystems of Ethiopia (EIBC, 2007). During the assessment time conducted in this study period, it was confirmed that these plant species are common in different wetlands of Ethiopia such as Lake

Tana, Lake Awassa, Lake Zeway, and in different Lakes found in Debera-Zeyet (Birhanu, 2007). The seedlings are taken from JWBO CW site and Zeway for the pilot scale SSFCW.

It is also common in most parts of Ethiopia as ornamental plant in cities such as Addis Ababa, Bahir-Dar, and Jimma and in natural wetlands found in different parts of the country. These palm attracts many birds particularly pink breasted pigeon to nest in it (Sebsebe et al., 1997;EIBC, 2007). In addition to these, this plant is used by the local community to make a variety of decorative handicrafts, such as baskets in different parts of Ethiopia as studied by Afework (2006). Though the socio-economic advantages of these wetland plant species are well known, their potential for wastewater treatment is not well understood. For instance, only few studies investigated the potential of *Cyperus papyrus*, (which colonizes many wetlands in Africa), for wastewater treatment (Birhanu, 2007).

2.11 Abiotic factors and their influence on wetlands

2.11.1 Dissolved oxygen, PH, temperature

Oxygen in wetland systems is important for heterotrophic bacterial oxidation and growth. It is an essential component for many wetland pollutant removal processes, especially nitrification, decomposition of organic matter, and other biological mediated processes. Nitrification and microbial respiration for the breakdown of organic matter depend on the DO concentration in water. The loading rate of organic matter should not exceed the aeration capacity of the wetland (Al-omari & Fayyad, 2003). Thus; the minimal required DO concentration to assure an efficient removal of organic matter depends on the loading rate of the organic matter. The higher the loading rate, the higher the DO must be (Al-omari & Fayyad, 2003). CWs often have both aerobic and anaerobic sections in their bed. The top section can still be aerobic as oxygen can penetrate through the pores. For CWs with a deep bed, the DO can be improved by using aquatic plants with a large root system (Rai et al., 2013).

Wetland waters usually have a pH of around 6-8 (Kadlec & Knight, 1996). The biota of wetlands especially can be impaired by sudden changes in PH.

According to Rob (2010) Temperature is a widely fluctuating abiotic factor that can vary both diurnally and seasonally. Temperature exerts a strong influence on the rate of chemical and biological processes in wetlands, including BOD decomposition, nitrification and denitrification. In cold climates, accumulated litter serves as thermal insulation and plant species selection may be more important than in warmer climates. Some species provide a good cover and insulation during winter (Rob et al., 2010).

2.12 Influence of operational parameters on pollutant removal

2.12.1 Organics

Effluent organic loading (BOD₅ and COD) for both SF and SSF treatment wetlands showed strong positive correlation with influent organic loading. This indicates that the residual concentration of BOD in the effluent of most treatment wetlands was influenced by the inlet concentration. Vymazal (2002) reported similar findings for 44 horizontal systems in the Czech Republic. Influent organic loading proved to influence wetland performance as reflected in effluent residual organics. The explanation is that up to an optimum loading, removal efficiencies increase as loading increases and correlate positively with mass loading rates. To illustrate, Ghermandi et al, (2010), reviewed 25 tertiary surface flow treatment wetlands and found that in 19 cases, effluent BOD were below 10 mg/l when the systems were not overloaded in terms of influent organic loading, where as in three cases, effluent exceeded 30 mg/l of which two were caused by higher than recommended influent BOD loading. Results from other operational wetlands for different types of wastewater have also exhibited a similar effect of influent loading on effluent residual BOD concentrations (Frazer & Ronnie, 2010).

In addition, correlation results indicate hydraulic and pollutant loading strongly influence wetland performance for organic removal (BOD, COD). In all cases, removal generally decreases as loading increases. Hydraulic loading was not found to correlate with nutrient removal. Overall, it can be concluded that organic removal can be modeled better compared to nutrient in treatment wetlands. Since removal of solids and coliforms are not primarily influenced by the key design parameters, it is expected that they will fit into any design model

develop (Frazer & Ronnie, 2010).

2.12.2 Microorganisms

As explained on hand book of constructed wetland (vol 1) a fundamental characteristic of wetlands is that their functions largely regulated by microorganisms and their metabolism (Wetzel, 2001). Microorganisms include bacteria, yeasts, fungi, protozoa, rind algae. The microbial biomass is a major sink for organic carbon and many nutrients. Microbial activity include transforms a great number of organic and inorganic substances into innocuous or insoluble substances, Alters the reduction/oxidation (redox) conditions of the substrate and thus affects the processing capacity of the wetland, is involved in the recycling of nutrients. Some microbial transformations are aerobic others are anaerobic. Many bacterial species are facultative anaerobes, that is, they are capable of functioning under both aerobic and anaerobic conditions in response to changing environmental conditions.

Microbial populations adjust to changes in the water delivered to them. Populations of microbes can expand quickly when presented with suitable energy-containing materials. When environmental conditions are no longer suitable, many microorganisms become dormant and can remain dormant for years (Hilton, 1993).

Toxic substances, such as pesticides and heavy metals, can affect the microbial community of a constructed wetland and care must be taken to prevent such chemicals from being introduced at damaging concentrations (Florentina & John, 2014). However according to handbook of constructed wetland, the limited capacity of natural systems for nutrient removal is an advantage when the treatment goal is to produce a reclaimed wastewater for irrigation to promote plant growth. A low pathogen concentration, high nutrient content (in particular N) and low presence of heavy metals or other toxic pollutant in reuse water are very desirable in reclaimed wastewater for agricultural irrigation (Florentina & John, 2014).

Therefore, in this study the vertical subsurface constructed wetland (VSSFCW) wastewater treatment system evaluated for the treatment of a highly contaminated river with non-point source pollutant, of Akaki River, generated due to open discharge of different types of

wastewater to the natural river without any treatment. The study was held on a pilot scale operation of SSFVCW treatment system. The goal is to investigate the efficiency of the treatment system in order to produce reasonable quality reclaimed water appropriate for agricultural irrigation claiming the coupled benefits of water and nutrient recycling.

2.13 Processes in subsurface flow constructed wetlands

Dierberg et al., (2002) Discussed wetlands have been considered an effective means to treat point source and non-point source of nutrients and improve water quality in downstream lakes and rivers. The benefits of using wetlands to treat non-point source of pollutants include the ability to operate under a wide range of hydraulic loads, provide internal water storage capacities, and remove or transform contaminants.

2.13.1 Biological processes

There are six major biological reactions involved in the performance of constructed wetlands, including photosynthesis, respiration, fermentation, nitrification, denitrification and microbial phosphorus removal. Wetland plants perform photosynthesis, with the process adding carbon and oxygen to the wetland. Both carbon and oxygen drive the nitrification process. Respiration is the oxidation of organic carbon, and performed by all living organisms, leading to the formation of carbon dioxide and water. The maintenance of optimal conditions in the system is required for the proper functioning of wetland organisms. Fermentation is the decomposition of organic carbon in the absence of oxygen, producing energy-rich compounds (e.g., methane, alcohol, volatile fatty acids). This process often undertaken by microbial activity. Nitrogen removal by nitrification/denitrification is the process mediated by microorganisms (Mitchel & Mcnevin, 2001). The study by Mitchel and Mcnevin (2001) summarize wetland microorganisms, including bacteria and fungi, as which remove soluble organic matter, coagulate colloidal material, stabilize organic matter, and convert organic matter into various gases and new cell tissue. Rob (2010) further describes that many of the microorganisms are the same as those occurring in conventional wastewater treatment systems. Different types of organisms, however, have specific tolerances and requirements for dissolved oxygen,

temperature ranges and nutrients.

2.13.2 Chemical processes and physical processes

The chemical processes in wetland include different mechanisms. Metals can precipitate from the water column as insoluble compounds. Exposure to light and atmospheric gases can break down organic pesticides, or kill disease-producing organisms (EPA, 1995). The pH of water and soils in wetlands exerts a strong influence on the direction of many reactions and processes, including biological transformation, partitioning of ionized and un ionized forms of acids and bases, cation exchange, solid and gases solubility.

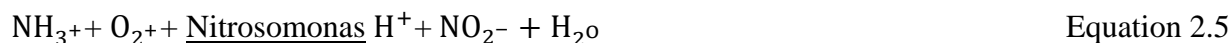
Sedimentation and filtration are the main physical processes leading to the removal of wastewater pollutants. The physical processes responsible for removing suspended solids include sedimentation, filtration, adsorption onto Biofilm and flocculation/precipitation. With low wetland water velocities and appropriate composition of influent solids, suspended solids will settle from the water column within the wetland. Sediment resuspension not only releases pollutants from the sediments, it increases the turbidity and reduces light penetration. The effectiveness of all processes (biological, chemical, physical) varies with the water residence time (i.e., the length of time the water stays in the wetland). Longer retention times accelerate the remove of more contaminants, although too-long retention times can have detrimental effects (Kenneth, 2001).

2.13.3 Wetland nitrogen processes

The most important nitrogen species in wetlands are dissolved ammonia (NH_4^+), nitrite(NO_2^-), and nitrate(NO_3^-). Other forms include nitrous oxide gas (NO_2), nitrogen gas (N_2), urea (organic), amino acids and amine (Kadlec & Knight, 1996). The various nitrogen forms are continually involved in transformations from inorganic to organic compounds, and vice-versa. Many of these transformations are biotic, carried out by nitrobacter and nitrosomonas (Kadlec & Knight, 1996). As it undergoes its various transformations, nitrogen is taken up by wetland plants and microflora (preferentially as NH_4^+ or NH_3) some is leached to the subsoil, some is liberated as gas to the atmosphere, and some flows out of the wetland, normally in a dissolved form. Organic nitrogen

comprises a significant fraction of wetland biota, detritus, soils, sediments and dissolved solids (Kadlec & Knight, 1996). It is not readily assimilated by aquatic plants, and must be converted to NH_4^+ or NH_3 through multiple conversions requiring long reaction time (Kadlec & Knight, 1996). The process of biological nitrogen removal follows several sequences such as Nitrification first takes place, generally in the rhizosphere and in biofilms (aerobic process). Denitrification may then follow, occurring in soils and below the oxidized microzone at the soil/water interface, as it is an anaerobic process (Broderick et al., 1989). Nitrification is a two-step process catalyzed by Nitrosomonas and nitrobacter bacteria as Equation 2.5 and 2.6 respectively.

In the first step, ammonia is oxidized to nitrite in an aerobic reaction catalyzed by Nitrosomonas bacteria.



The second step, Nitrobacter bacteria oxidize the nitrite produced aerobically forming nitrate.



The first reaction produces hydroxonium ions (acid pH), which react with natural carbonate to decrease the alkalinity (Mitchel & Mcnevin, 2001). In order to perform nitrification, the nitrosomonas must compete with heterotrophic bacteria for oxygen. Moreover, according to Reed et al. (1995) The BOD of the water must be less than 20mg/l before significant nitrification can occur. Temperatures and water retention times also may affect the rate of nitrification in the wetland (Rob, 2010).

Denitrification is the process in which nitrate is reduced in anaerobic conditions by the benthos to a gaseous form. The denitrifying bacteria Pseudomonas and other bacteria catalyze the reaction. Denitrification requires nitrate, anoxic conditions and carbon sources (readily biodegradable). Nitrification must precede denitrification, since nitrate is one of the prerequisites. The process of denitrification is slower under acidic condition. At a PH, 5-6, N_2O produced. For a PH below five, N_2 is the main nitrogenous product. NH_4 Is the dominant form of ammonia-nitrogen at a pH of seven, while NH_3 (present as a dissolved gas) predominates at a pH of 12. Nitrogen cycling

within, and removal from, the wetlands generally involves both the translocation and transformation of nitrogen in the wetlands. This process includes sedimentation (resuspension), diffusion of the dissolved form, adsorption/desorption of soluble nitrogen to soil particles, organism migration, assimilation by wetland biota, seed release, ammonification (mineralization) (Orga-N – NH_4^+), ammonia volatilization (NH_4^- – NH_3 (gas)), bacterially mediated nitrification/denitrification reactions, nitrogen fixation ($\text{N}_2, \text{N}_2\text{O}$ (gases – organic-N)), and nitrogen assimilation by wetland biota (NH_4^+ , NO_x organic – N, with NO_x usually as NO_3^-). Precipitation is not a significant process due to the high solubility of nitrogen, even in inorganic form (Kadlec & Knight, 1996).

2.13.4 Phosphorus removal

Phosphorus removal in wetlands is based on the phosphorous cycle, and can involve a number of processes. Primary phosphorus removal mechanisms include adsorption, filtration and sedimentation. Other processes include complexation/precipitation and assimilation/uptake. Particulate phosphorus is removed by sedimentation, along with suspended solids. Reddy et al (1989) further found other Phosphorus removal mechanism from the wastewater, which is by plant uptake, microbial assimilation, precipitation with cations (Ca, Mg, Fe, Mn) and adsorption onto clay and organic matter. Therefore, the configuration of constructed wetlands should provide extensive uptake by Biofilm and plant growth, as well as by sedimentation and filtration of suspended materials. Phosphorus is stored in the sediments, biota, (plants, Biofilm and fauna), detritus and in the water. The interactions between compartments depend on environmental conditions such as redox chemistry, pH and temperature. The redox status of the sediments (related to oxygen content) and litter/peat compartment is a major factor in determining which phosphorus cycling processes will occur. Under low oxygen conditions (low redox potential), phosphorus is liberated from the sediments and soils back into the water column, and can leave the wetland if the anaerobic condition is not reversed (Moss et al., 1986). In addition, recent findings made by Jianbo et al (2011) confirm that microbial P-consumption is higher in aerobic regions of the rhizosphere, whereas P-mineralization is predominant in the anaerobic regions. On the other side, Taylor & Francis (2015) discussed that the removal of phosphorus is dependent on the kind of substrate used. Vymazal (1996) Further explains that Substrate containing high Ca, Al

and Fe will have high phosphorus removal efficiency the higher $PO_4 - P$ treatment efficiency of some constructed wetlands was probably due to the high amount of iron rich sand in the substrate.

2.13.5 Pathogen removal

Wetlands are very effective at removing pathogens, typically reducing pathogen number by up to five orders of magnitude from wetland inflows (Reed et al., 1995).

The processes that may remove pathogens in wetlands include natural die-off, sedimentation, filtration, ultra-violet light ionization, unfavorable water chemistry, temperature effects, predation by other organisms and PH. Kadlec and Knight (1996) further showed that vegetated wetlands seem more effective in pathogen removal, since they allow a variety of microorganisms to grow which may be predators to pathogens.

2.13.6 Heavy metal removal

According to Rob (2010) there are three main wetland processes that remove heavy metals; namely, binding to soils by sedimentation and particulate matter, precipitation as insoluble salts, and uptake by bacteria, algae and plants (Kadlec & Knight, 1996). These processes are very effective, with removal rates reported up to 99% according to Reed et al, (1995).

A range of heavy metals, pathogens, inorganic and organic compounds present in wetlands can be toxic to biota. The response of biota depends on the toxin concentration and the tolerance of organisms to a particular toxin. Wetlands have a buffering capacity for toxins, and various processes dilute and break down the toxins to some degree (Rob, 2010). However, heavy metal removal efficiency of Vertical subsurface constructed wetland is not in the scope of this study.

2.14 Related outcomes of constructed wetlands

There are several direct human benefits possible from constructed wetlands. The improvement of water quality by wetlands has been found to benefit human health by reducing disease-causing bacteria and viruses (Jing *et al.*, 2001).

Wetlands remove toxic chemicals found in wastewater in addition to nutrients. Harvesting of wetland vegetation has been used for the production of methanol (USEPA, 1999). Constructed wetlands with public access and public use provide recreation, research, and educational opportunities. Public education has ancillary benefits of generating support for water quality and watershed protection. Constructed wetlands have been used in combination with other treatment mechanisms to provide safe drinking water (Elias *et al.*, 2001).

Even though there are many benefits from constructed wetlands designed to treat water quality, these wetlands can also have detrimental outcomes. For example, the use of farmland to construct a wetland results in a loss of that land for farming or another land use. Constructed wetlands located in other water bodies (i.e., wetland, stream, or lake) or immediately adjacent to natural water bodies can negatively affect the natural water quality or quantity of these water bodies (USEPA, 2000).

3. RESEARCH METHODOLOGY

3.1 Description of the study area

3.1.1 Location of study area in Addis Ababa

Addis Ababa is the capital city of Ethiopia. It is a seat of the African Union (AU) and the United Nations Economic Commissions for Africa and gateway for diplomats and tourists. As per the Central Statistical Agency (CSA-2013) population projection, Addis Ababa has a total population of 3,353,000 with family size of 4.2 in 2016 and the total area of the city is 521.31 square kilometer (52,131hectare). Out of it, 22,000 hectares land designed for green frame, but currently it is estimated that 7,900 hectares of land is allocated for green frame. The average elevation of the city is about 2,057 meter above sea level (m.a.s.l). Which is the highest when one can compared with many cities of our world. In terms of governance, there are three level of government: City Government at the top, 10 sub city administrations in the middle and 116 woredas, which is the smallest unit of government in the hierarchy.

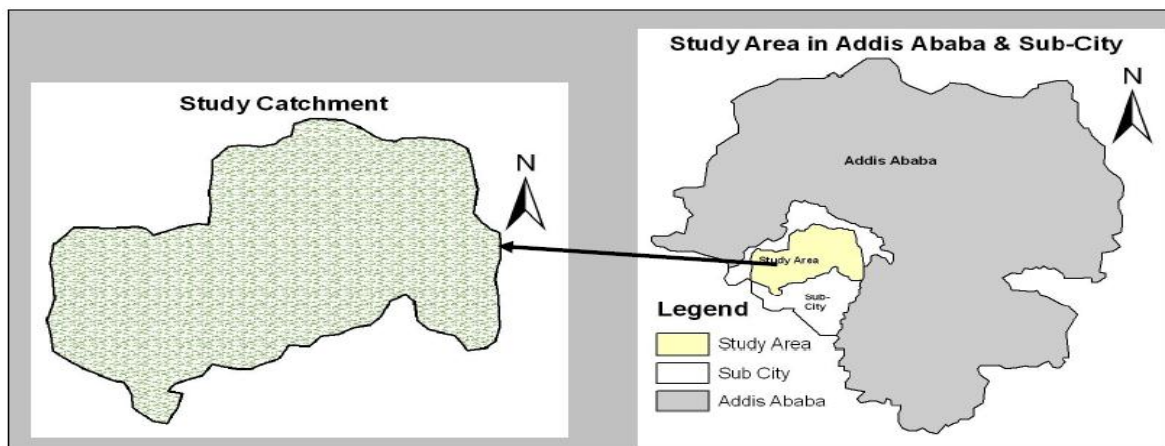


Figure 2. Location of the study area in Addis Ababa

The Akaki catchment have an average annual rainfall of 984.7 millimeters. The City has a tropical climate and two distinct seasons. During the study period the rainy season starts early from May and ends at late September and the dry season begins from October to March. With an average

humidity of 62%, temperatures range from with an average temperature of 20°C.

3.2 Description of pilot scale constructed wetland

The pilot scale Vertical subsurface flow constructed wetland (VSSFCW) was constructed to assess the feasibility of treating non-point source polluted wastewater from Akaki River. Within six-month experiment under the presence of three different kinds of plants, and the growth media was gravel with 45% porosity of the gravel bed. The larger gravel size advances longer run time by reducing maintenance cost.

The pilot VSSFCW is constructed inside the researcher residence compound for both monitoring and supervision purposes. Hence the wetland was Located at 8°59'27.30"N 38°41'22.87"E and an Elevation of 2374m in west Addis Ababa around Ayertena area.

The amount of wastewater influent to the pilot VSSFCW was determined based on the irrigation crop water requirement (CWR) data from cropwat model. Commonly irrigated crops on the Akaki River side urban agriculture are cabbage, lettuce, tomato. The CWR result for this crops when irrigated for 24 hours is 0.24 l/s/h and 0.48 l/s/h for 12 hours irrigation on of 0.6-hectare farmland. This is equivalent to 25 m³/d . Then the very small portion of this volume of water is taken for design purpose (i.e. 1/250 th hand) which is 0.1m³/d . Then based on this design discharge the other wetland parameters are calculated.

First-order degradation represents the basic design equation (Equation 3.1) employed widely for subsurface constructed wetland. At present the state-of-the-art [First-order models] k- C* model seems to be the best available design tool and many researchers use this model for the designing purposes as it was discussed in Chapter 2 section 2.6.5.1. The design of vertical subsurface wetlands was carried out with this equation for BOD, TSS, TN and TP removal, since the other parameters are not affected by the design parameters.

$$\left(\frac{C_{\text{out}}-C^*}{C_{\text{in}}-C^*}\right)=e^{(-K_A/q)} \quad \text{Equation 3.1}$$

$$q = Q/A \quad \text{Equation}$$

3.2

$$K_{A,T} = K_{A,20} \theta^{(T-20)} \quad \text{Equation}$$

3.3

A=required area (h), Q=water flow rate (m³/d), kA is the areal decomposition constant (m/d), q=hydraulic loading rate (m/yr.), C* = background concentration (mg/L), C_{in} inflow concentration (mg/L), C_{out} outflow concentration (mg/L), K_A is the temperature-dependent first-order areal rate constant at temperature of T, K_{A,20} is the first-order areal rate constant at 20°C, θ is the design parameter, the values of K_{A,20} and θ at 20°C are given in Chapter 2, Section 2.6.5.1 .

The hydraulic residence time (HRT) in the wetland is calculated using Equation 3.4 (Reed et al, 1995).

$$HRT = \frac{LWYn}{Q} \quad \text{Equation}$$

3.4

n = effective porosity media, % as a decimal, L = Length of the bed (m), W = Width of the bed (m), Y = Average depth of liquid in bed (m), Q = the average of the inflow and outflow (m³/d).

The flow (Q) is the arithmetic average between the inflow and the outflow. However, as reported by Zhang (2012), it is usually reasonable for a preliminary design estimate to assume that outflow equals inflow.

Combining Equation 3.1 and Equation 3.2 gives Equation 3.5, by which the area of the wetland was calculated.

$$A_w = \left(\frac{0.0365 \cdot Q}{K_A} \right) \times \ln \left(\frac{C_{in} - C^*}{C_{out} - C^*} \right) \quad \text{Equation 3.5}$$

A_W is the required wetland area (ha) and Q is the water flow rate (m^3/d).

During calculation of the pilot VSSFCW area (A_W), the design flow (Q) used was $0.1m^3/d$; the Influent Concentration (C_{in}) was found from a previous wastewater quality test report on Akaki River; and Target Effluent Concentration (C_{out}) was found from WHO standard of the nutrients for Vegetables likely to be eaten raw for effluent reuse. According to Environment (2000), the area of the constructed wetland will be the larger area among the calculated areas needed to eliminate the corresponding nutrient. The calculation results in table 2 show the proposed pilot scale VSSFCW size for this research is about $1.5 m^2$, which is the area, required to reduce the nutrient concentration of total phosphorous (TP) from the influent wastewater.

Table 2. Pilot VSSFCW Area result

Nutrient Type in mg/l	TSS	BOD	TN	TP	NH_3
Influent Concentration(C_i)	321	100	15.5	24	7.5
Target Effluent Concentration(C_e)	35	35	10	14.5	5
Wetland background limit, mg/L(C^*)	$7.8 + 0.063C_i^*$	$3.5+0.053C_i^*$	2	0.05	0
Calculated pilot VSSFCW Area	0.1	1.3	0.7	1.5	0.6

* Adopted from Preliminary model parameter values for Kadlec and Knight's design equations (Kadlec and Knight, 1996)

3.2.1 Wetland Configuration

VSSFCW consists of Inflow tank, porous medium, bed planted with macrophytes and effluent tank. The influent tank receives the wastewater from the sources of production and gradually discharges it into the porous medium of the bed. The main part of a SSFCW is the bed, which consists of the porous media and the plants. The effluent tank receives the treated water and stores until discharged into the environment (Peter & Alette, 2014).

The data analysis presented in this research is based on the pulse-loaded, unsaturated vertical flow wetlands. The operational regime of pulse-loading, followed by rest periods, allows Vertical flow

(VF) systems to operate in a mode of unsaturated flow, which allows the introduction of air (and oxygen) into the VF bed (Kadlec & Wallace, 2009). Consequently, VF wetlands are more amenable to aerobic reduction of organic matter and associated BOD.

3.2.2 Pilot VSSFCW Set-Up

After fixing area of Pilot VSSFCW, other wetland dimensions are calculated to be 2.1m x 0.7m x 0.3m (L x W x H) and have a slope in the bottom of 1 %. This is according to widely applicable VSSFCW aspect ratio (L: W) 3:1 as discussed in Chapter 2 Section 2.10.6. It is recommended that an adjustable outlet and slightly sloped bottom (< 1%) for VFSSCW (EPA, 2007). To achieve active treatment depth in the pilot plant a depth of 0.3m taken as explained by Chinniah et al., (2000). Total depth of the growth media including freeboard was 0.4 m and the sewage level was 0.30 m below the gravel surface.

During the civil work, Pilot VSSFCW basin constructed from concrete hollow block according to the dimensions. The inside face of the basin was well plastered in order to prevent seepage to the ground. Please see figure 3. Also, I was watered daily for a week to make it strong. An impermeable plastic placed in the wetland basin to avoid seepage.



Figure 3. Plastered and impermeable plastic cover of pilot plant

A suitable gravel type was assessed and used during the research. During the assessment of this research, it was investigated that in the existing Jehovah's Witnesses Branch Office (JWBO)

subsurface constructed wetland gravel-having diameter of 40-80mm was used to avoid easily clogging of the subsurface constructed wetland. This experience was adopted for the pilot scale constructed wetland herein to avoid easily clogging of the wetland. Since constructed wetland in this study is aimed to be operated by farmers on site; it should have to reduce maintenance cost.

Kadlec & Wallace (2009) reported general mean porosity of a clean gravel media for subsurface wetlands in the range 0.30–0.45 %. Considering the future decline in bed hydraulic conductivity, this research uses 45% porosity to calculate HRT of the wetland. Also, using porosity equation, the appropriate gravel size calculated was 50mm. Please see Equation 3.6. Sieve analysis was made by sieving the selected gravel between the sieves of 37 mm & 50mm. To remove some dirt, the gravel was washed. Please see figure 4. Then the pilot VSSFCW was filled with gravel with a particle size of 37-50 mm.



Figure 4. Sieve analysis and washed gravel

$$\text{Porosity} = ((\text{Total Volume} - \text{Volume of the Solid}) / \text{Total Volume}) \times 100\% \quad \text{Equation 3.6}$$

According to the design flow, 0.1m³ elevated tank was placed near the Pilot VSSFCW basin. The wastewater is transported from Akaki River of the selected location in the catchment near the intensive urban agriculture area around Germen Bride, using plastic containers. The wastewater was filled into the elevated tank and allowed to pass through four PPR perforated pipes, which are

laid down horizontally on the top of wetland gravel bed. An outlet valve controls the influent from the tank to the perforated pipes. When the valve is opened, raw sewage influent is distributed vertically from the top of the unit through horizontally laid perforated pipes on the top of the gravel bed by gravity. The diagram of the experimental installation and the existing pilot plant are presented in figure 5.

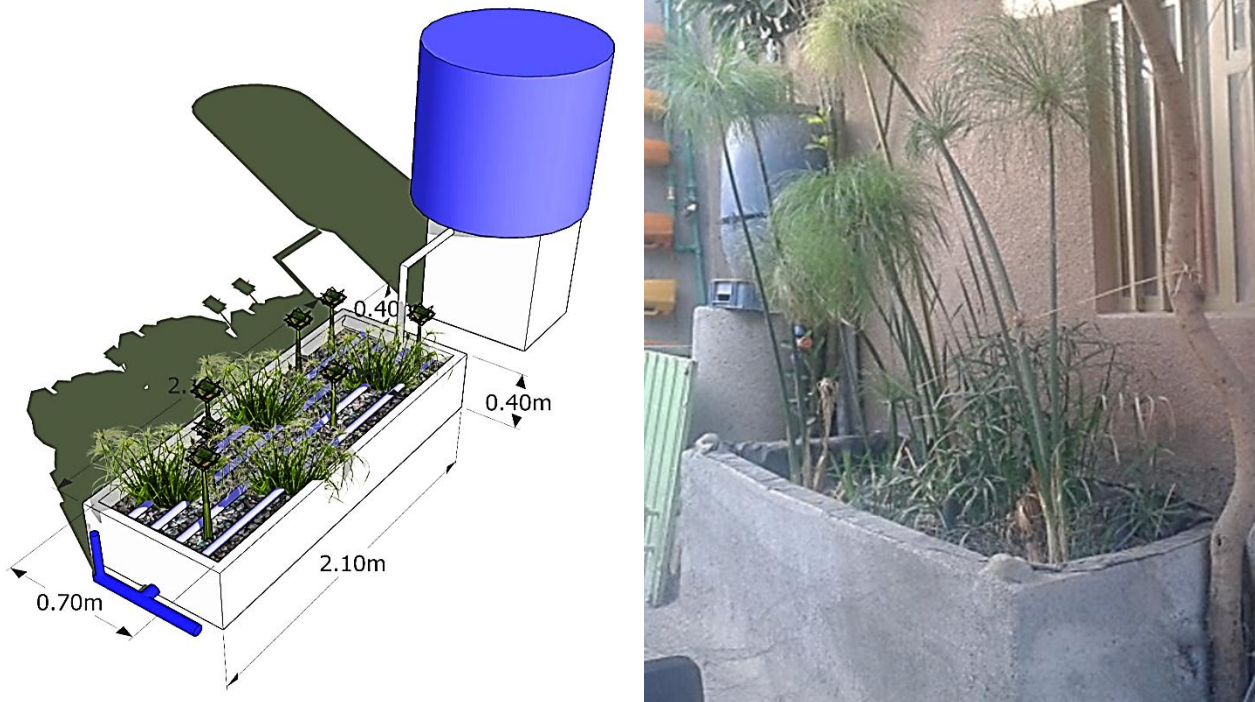


Figure 5. Pilot VSSFCW experimental installation

Finally, the treated effluent collected from the bottom of the unit after the designed HRT. The calculated HRT in this study is 3 days, however, the HRT has to be long enough to allow the removal processes to take place as discussed in Chapter 2 Section 2.10.1, hence HRT of 8 days is adopted. The HRT was controlled using an outlet valve.

Since the flow is plug flow type or batch flow, the effluent is collected on the eighth day by opening the outlet valve.

At the out let of the wetland there is outlet valve in order to discharge the effluent at the right time (HRT). There is also water level monitoring pipe; please see figure 6.. The outlet structure



design increases the treatment efficiency of the wetland, since the effluent falls down from raised outlet pipe; aeration takes place and



receives the final treatment. Please see figure 7.

Figure 6. Inlet and outlet structures of pilot VSSFCW

Figure 7. Effluent collection

The bed runs with the calculated hydraulic application rate and hydraulic retention time. Filter

media layers of the bed were distributed equally. Influent distribution is regular through distribution system (perforated PPR pipes) to avoid dead zone in the pilot VSSFCW. Within perforated pipes, wastewater flows slowly through filter layers to roots of plants. Please see figure 8. The effluent collected by filtered PVC pipe placed at the bottom of the bed. The water level in the wetland is maintained below the gravel, to avoid flooding of the macrophytes and facilitate oxygen supply of the wetland system through flow-regulating valve. There are vertical pipes extending above the surface of the wetland bed. These pipes used as inspection pipes in order to control the water level as well as for the ventilation of this wetland system.



Figure 8. Influent distribution of the Pilot VSSFCW

3.2.3 Planting

In this study the pilot VSSFCW, was planted with three plant species, namely *Cyprus papyrus* (Papyrus), *Cyprus alternifolia* (Umbrella plant) and Bruised Reed as shown in Figure 9. Mixed plant species help to maximize root biomass in the wetland substrate that in turn results in aerobic degradation around the root zone and nutrient removal (Brix, 1994; Oketch, 2003).

Young macrophytes such as *Cyprus papyrus* (Papyrus), Bruised Reed and *Cyprus alternifolia* collected from JWBO. The plants were cut to a length of about 30 cm including stem and rhizome and were grown in the bed. The density of macrophytes was maintained at about 20 plants per m^2 . After sowing, the plants were irrigated with fresh water to ensure their rhizome can

acclimate to the new environment. After 20–30 days, fresh water was replaced by diluted wastewater to supply nutrients to macrophytes. After one month, the diluted wastewater was replaced by wastewater that was mainly used for the period of experiment. After the first two months of experiment, both *Cyprus papyrus* (Papyrus), and *alternifolia* (Umbrella plant) showed the best survival and adaptation as shown by rapid growth of their leaves and stems. On the other hand, Bruised Reed showed less vigor at the first two months, but afterwards, they showed satisfactory growth rate. Please see figure 9 and figure 10.

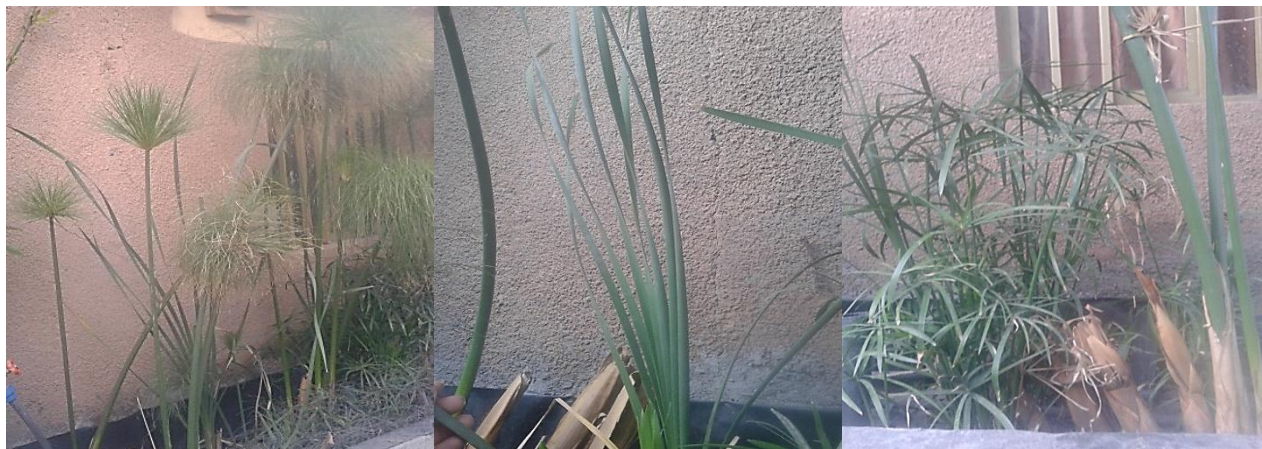


Cyprus papyrus (Papyrus),

Bruised Reed

Cprus alternifolia

Figure 9. Pilot scale VSSFCW planted bed; young macrophytes



Cyprus papyrus (Papyrus)

Bruised Reed

Cprus alternifolia

Figure 10. Pilot scale VSSFCW planted bed; fully grown macrophytes

3.2.4 Experimental set-up

A detailed influent and effluent characterization of all collected samples were carried out for selected physicochemical and bacteriological wastewater quality parameters. The common quality parameters determined were BOD₅, COD, TSS, Ammonia, Nitrate, Nitrite, Total Nitrogen, Total Phosphorus, Sulfide, Fecal coliform, PH and Temperature.

Sampling and analysis began in August, 2018, i.e., 2 months after sowing, and continued until end of February, 2019. Samples were taken from the influent and effluent of the bed for four months throughout the study period. All samples were stored and analyzed in accordance with the Standard Methods for Examination of Water and Wastewater (Vu1, 2011).

To evaluate the performance efficiency of pilot VSSFCW, inlet grab samples were collected 8 days before the outlet ones, according to the hydraulic retention time of the wetland. During the first sampling, the river was diluted due to the heavy rainfall around the end of August 2018. However, after October 2018, i.e., during the dry season, the river shows the most un pleasant character. The river became black in color and had bad odor. Please see figure 11.



Figure 11. Akaki River during high rainy and dry season

3.3 Laboratory analysis

Laboratory examination was carried out at Environmental protection Agency, but total nitrogen and total phosphorus was examined at Ethiopian Construction Design & Supervision Works laboratory, due to the absence of the reagents for these parameters at Environmental protection Agency.

BOD₅ was determined using pressure sensor. The measurement heads close the analysis bottles hermetically, and the pressure changes are measured with pressure sensors and converted to BOD. These methods allow the measurement of the BOD in samples contaminated by high levels of carbon compounds without making dilutions because of the large pool of oxygen trapped in the bottle. The sample bottles are filled with a measured volume of sample. The microorganisms degrade organic substances using the gaseous oxygen trapped in the closed bottle. The carbon dioxide formed by this process is absorbed, generally with sodium hydroxide pellets. The pressure changes of each day for five days are measured by a manometer and converted to oxygen consumption by the device to estimate the BOD value.

COD was measured according to Reactor Digestion Method USEPA approved for reporting wastewater analysis. Well-mixed sample is added in to a 250mL beaker and stirred slowly with a magnetic stir plate. This step Invert a test vial several times to mix. Then using a pipet 2.0 mL of sample is dropped to the test vial. Invert gently several times to mix. The vial got very hot during mixing. Then the vial was inserted in the preheated DRB200 reactor and kept in the reactor for 2 hours. After two hours and invert gently several times while the vial was hot. The temperature of the vial was decreased to room temperature. Finally, the vial was inserted into the cell holder. DR 1900 Push READ. Results show COD in mg/L.

Ammonia (NH₃) was measured through Salicylate method. In diluent reagent vial, a 2 ml of sample and Salicylate reagent powder to 5 ml of the vial was added. After 20 minutes reaction

period a green color was developed indicating the presence of NH_3 . Blank sample was prepared by deionized water.

Cadmium Reduction Method using Powder Pillows as reagent was used to measure Nitrate(NO_3). 10 ml of sample wastewater was taken, then NitraVer 5 nitrate reagent powder pillow was added to the sample. Then the sample was shaken vigorously for one minute to dissolve the powder. After five minute reaction period, an amber color was developed indicating the presence of NO_3 . Blank sample was prepared by 10 ml of the sample. The blank sample cell was used to calibrate the instrument to display 0.0 mg/l NO_3 . Finally, the sample was cleaned and inserted in to the instrument holder and read for NO_3 mg/l. The instrument displayed the concentration of NO_3 in the sample.

Nitrite(NO_2) was measured by Diazotization method using Powder Pillows as reagent. The same procedure was followed for nitrite as nitrate, except using NitraVer 3 nitrite reagent powder pillow was added to 25ml of the sample and blank sample was prepared by distilled water. After reaction time of 20 minutes, pink color was developed indicating the presence of NO_2 .

Total Suspended Solids and turbidity were measured calorimetrically by spectrophotometer (DR/3000 and DR 5000, USA respectively) according to HACH instructions. A well shaken 100ml sample from the wastewater was taken as a representative sample. On another sample cell, a blank sample was prepared from distilled water. The spectrophotometer was calibrated then the sample was measured.

Environmental parameters PH and water temperature were measured on all samples due to the importance of these factors for microbial growth and metabolism (Gray 2004). Temperature and pH was measured using pH and Temperature conductivity meter. The sample was poured in the beaker and the probe inserted into it. Readings were then taken when they were stable.

Sulfide(S^{2-}) was measured according to methylene blue method. Sample was added to a sample cell using pipet. To prevent sulfide loss, the sample was not mixed more than necessary. The blank sample was prepared with deionized water. For both samples, 10 mL for spectrophotometers and 25 ml for colorimeters were used. Sulfide 1 and 2 Reagents were added

to each sample cell, 0.5 ml for spectrophotometers and 1 ml for colorimeters; and spinned to mix. Initially, a pink color was developed indicating the presence of sulfide; and the solution became blue. After five minute reaction period, the blank sample was inserted into the instrument cell holder to adjust to display 0 $\mu\text{g/l}$ or 0.00 mg/l S^{2-} . Finally, the prepared sample inserted and the results displayed in $\mu\text{g/l}$ or mg/l S^{2-} .

The bacteriological quality indicators, Total Coliform (TC) and Fecal Coliform (FC) were evaluated using Membrane Filter (MF) procedures of standard method for the examination of water and wastewater (APHA, 1998). Samples were serially diluted (10^1 to 10^6) using distilled water and 100 ml of the diluted water was filtered through filter paper in order to retain bacteria using filtering unit. Then a filter paper, with a pore size $0.45\mu\text{m}$, was placed on a surface of absorbent pad socked with Membrane Lauryl Sulfate Broth medium and incubated at 37°C for Total Coliforms and 44°C for Fecal Coliforms. Yellow Colonies are atypical colony characteristic of TC and FC (APHA, 1998). After 24 hours, Yellow Colonies were counted using colony counter and the results were recorded as the number of Colony Forming Unit (CFU) of TC and FC per 100 ml.

Total Nitrogen was determined by Semi-Micro Kjeldal distillation method. In which a well-mixed waste water sample was digested in a 500 ml flask in a Hook at 300°C for 30-40 minutes with addition of 10ml copper sulphate(CuSO_4). Then, NaOH & $\text{NO}_2\text{S}_2\text{O}_3$ were added in to the digested sample. 20ml of 2% boric acid and 3 drops of an indicator was added in to another empty round flask. The sample was distilled with steam for 3 min please see figure 12 a. The Nitrogen was distilled to the other flask and trapped by the scavenger boric acid in the flask. Then, the distillate was titrated with (H_2SO_4), with normal concentration of (0.02N).

Total Phosphorus was measured calorimetrically by spectrophotometer (DR/ 2000, USA) according to HACH instructions please see figure 12 b. Well-mixed wastewater sample of 50 ml was digested in a 500 ml amber flask on a hot plate for 30-40 minutes, with addition of one drop of phenolphthalein, 1 ml of (H_2SO_4) and 0.4 ml ammonium persulfate ($(\text{NH}_4)_2\text{S}_2\text{O}_8$). This was continued until 10 ml of the sample remained. Then the sample was cooled and diluted up to 100 ml. Finally, pipet 50 ml sample into a clean test tube and add 8ml combined reagent and mix

thoroughly. After 10 minutes, but not more than 30 minutes, the absorbance of each sample at 880 nm wavelength on spectrophotometer using reagent blank as the reference solution was measured. Then, the absorbance was converted to concentration.



Figure 12. a. steam distillatory

b. spectrophotometer

3.3.1 Removal efficiencies

Since influent contaminant concentrations varied throughout the experimental period as shown in Table 3 of chapter 4, the removal efficiency of CWs cannot be determined just from effluent concentrations, as these depend on influent concentrations and follow a first order removal kinetics (Matamoros & Bayona, 2005). The pilot scale vertical flow constructed wetland removal efficiency were calculated using Equation 3.7 (Christon, 2004).

$$\text{Efficiency (\%)} = \frac{C_i - C_e}{C_i} * 100 \quad \text{Equation 3.7}$$

Where: C_i = is the concentration of the waste material in the influent (mg/l)

C_e = is the concentration of the waste material in the effluent

3.4 Modeling approach

In this study the modeling approach is mainly to determine the best process that contributes to the

total removal of Faecal coliform (FC) & optimizing the FC reduction in VSSFCW using model. The modeling tool used in this research was STELLA.

Modeling in this study is needed to know the best way to remove Faecal coliforms and the quantification of the contribution made by each process on the total removal of Faecal Coliform (FC) in order to get facts to optimize vertical sub surface flow constructed wetland system.

Processes that contributed to the total FC removal are inflow, outflow, sedimentation, filtration, and natural dying processes. Sedimentation is the separation from water, by gravitation settling, of suspended particles that are heavier than water (Metcalf and Eddy, 1991). Filtration was used extensively for achieving supplemental removal of suspended solids including particulate BOD from wastewater effluents of biological and chemical treatment processes (Metcalf and Eddy, 1991). The growth process was considered because within the system there was growth of microorganisms which occurred due to supply of nutrients. The death process might be due to unfavorable environmental conditions including UV from sunlight and temperatures unfavorable for cell reproduction (Polprasert, 2002).

3.4.1 Mathematical equations

The mathematical equations in this study are considered as sub models. They are adopted from the ones reported by Kimwaga *et al.* (2003), Scott (1995) and Metcalf and Eddy (1991) basing on the processes that took place during the total removal of FC in VSSFCW.

Inflow equation

$$FC_{ip} = \frac{Q_{in} \times FC_{in}}{V_{CW}} \tag{Equation 3.8}$$

Where FC_{ip} is FC concentration in the inflow process (Counts per 100ml per day), Q_{in} is the design flow rate (m³/d), FC_{in} is the concentration of FC (Counts per 100ml), V_{CW} is the volume of constructed wetland (m³).

Outflow equation

$$FC_{Op} = \frac{Q_o \times FC_o}{V_{cw}} \quad \text{Equation 3.9}$$

FC_{Op} is FC concentration in the out-flow process (Counts per 100ml per day), Q_o is the design flow rate (m³/d), FC_o is the concentration of FC (Counts per 100ml) in the outflow.

Sedimentation equation

$$FC_{sd} = \frac{FC_o \times s_r}{H} \quad \text{Equation 3.10}$$

FC_{sd} is the FC concentration in the sedimentation process (Counts per 100ml per day), s_r is sedimentation rate or settling velocity (m/d), H is the effective depth of the wetland (m).

Filtration equation

$$FC_f = \frac{FC_o \times f \times a \times b}{V_{cw}} \quad \text{Equation 3.11}$$

FC_f is the FC concentration in the filtration processes (Counts per 100ml per day), f is the filtration rate (m/d), a and b are constants in m.

Die-off equations

The death rate is usually a function of the viable population and environmental characteristics (Metcalf and Eddy, 1991). Also, Metcalf and Eddy (1991) outlined that the rate of disappearance of pathogenic bacteria and viruses due to die-off approximately follows first order kinetics.

$$r_B = k_B \times C_B \quad \text{Equation 3.12}$$

r_B is rate of bacteria die-off per unit time per unit volume of water (Count/L³.T), C_B is bacteria concentration (Count/L³) and k_B is die-off constant (T⁻¹), The k_B can be compared with the overall decay coefficient which presented by Kimwaga (2003).

$$r_Z = r_t \times r_i + r_f \quad \text{Equation 3.13}$$

r_Z is the overall removal rate coefficient (per day), r_t is the removal rate coefficient due to temperature (per day), r_i is the removal rate coefficient due to solar radiation (per day), r_f is the

removal rate coefficient due to combine effects of adsorption, filtration and sedimentation.

$$r_t = r_{20} \times \phi_T^{(t-20)} \quad \text{Equation 3.14}$$

(Arrhenius-van't Hoff equation)

r_{20} Is the removal rate at 20°C (per day), t is the water temperature 0C T, ϕ_T Is the temperature coefficient

$$r_i = \phi \times I_{avg} \quad \text{Equation 3.15}$$

ϕ Is the light mortality coefficient (m^2 per cal), I_{avg} Is an average solar radiation (cal per m^2 per day)

$$r_f = \frac{4}{\pi} n \alpha u \times \frac{(1-\theta)}{d} \quad \text{Equation 3.16}$$

n is single collector removal rate coefficient, α is the sticking efficiency, u is velocity of flow (m per day), d is the porous media grain diameter (m), θ is the porosity of the bed.

$$FC_d = r_z \times FC_o \quad \text{Equation 3.17}$$

FC_d is the FC concentration in the death process (FC counts/100ml.d), r_z is the overall removal rate coefficient (per day).

Growth rate equation

$$\mu = \mu_{max} \frac{S}{k_s + S} \quad \text{Equation 3.18}$$

μ is specific growth rate, (h^{-1}), μ_{max} is the maximum specific growth rate (time-1), S is the concentration of growth-limiting substrate in solution (mass/unit volume), k_s is half-velocity constant (mass per unit volume).

If bacteria growth depends on more than one substrate e.g. oxygen and ammonia ion then the equation would be as follows.

$$\mu = \left[\mu_{\max} \frac{S_o}{K_o + S_o} \right] * \left[\frac{S_{NH}}{k_{NH} + S_{NH}} \right] * \dots \quad \text{Equation 3.19}$$

S_o is O_2 concentration (mg/l), K_o is O_2 half-saturation constant (mg/l), S_{NH} is NH_3 - ammonia ion concentration (mg/l), k_{NH} is ammonia ion half-saturation constant (mg/l).

$$FC_g = \mu \times FC_o \quad \text{Equation 3.20}$$

FC_g is the FC concentration in the growth process (Counts/100ml.d), μ is specific growth rate, (d^{-1}), All the equations presented above were used in the FC model.

The quantification of the FC concentration was done by using STELLA modeling software. The equations presented in this section to represent the different processes are used as sub models in the main model. This situation is well presented in the model diagram as presented in figure 13.

3.4.2 Model simulation

The forcing functions in this study were DO, Temperature, pH and flow rates while the state variables are BOD_5 , NH_3 , and FC bacteria. The definitions of forcing function and state variable were explained modelling elements in chapter two. This modeling elements are clearly described in the diagram of STELLA model please see figure 13.

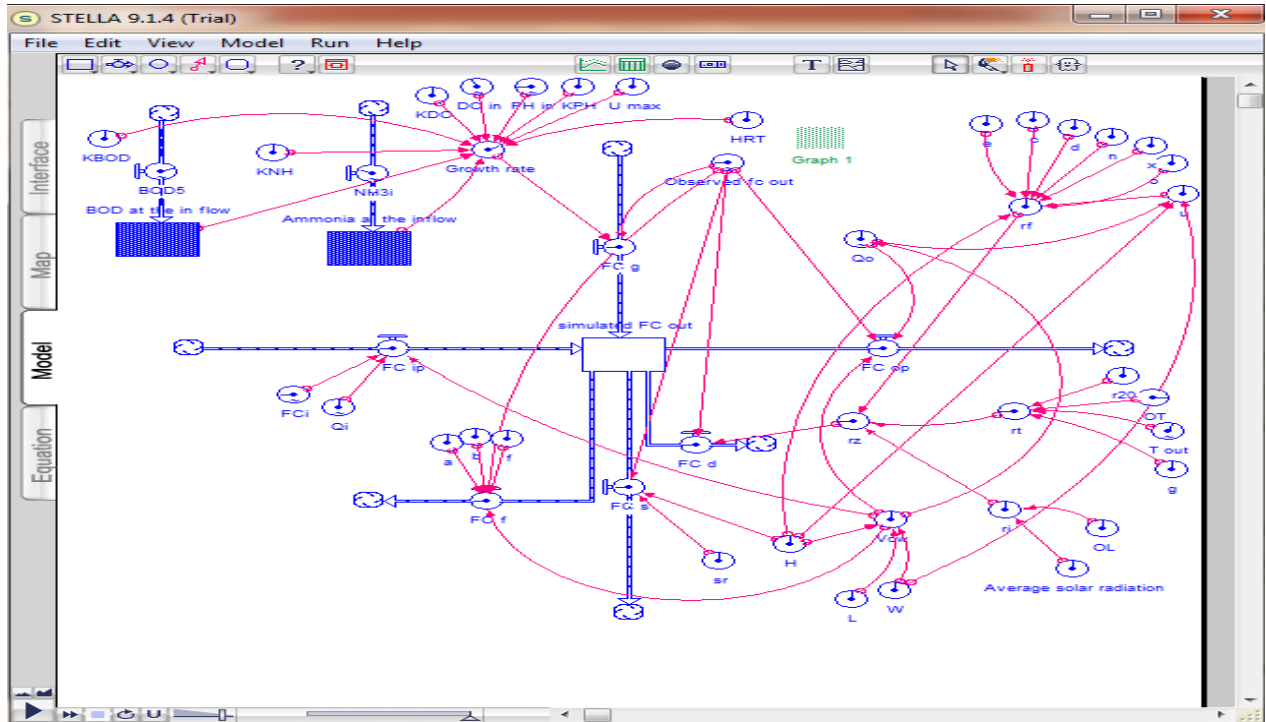


Figure 13. STELLA diagram for faecal coliform removal in VSSFCW

3.4.3 Model calibration

Calibration is the tuning of the model to fit the observed values. This involved varying the model parameters to obtain an optimal agreement between model calculations and the observed values and this was done based on adjusting the kinetic parameters.

In order to run model calibration there must be enough data. However due to laboratory problem and financial problems it was difficult to take more than few samples during the study period.

However by understanding the need to model calibrations, some secondary data was used. The data was on the Akaki river water quality on the selected parameters. This raw data was first used as an influent in the model then the effluent was calculated by The equations of Reed et al. (1995) based on the first-order plug flow as presented in Equation 3.21 and 3.22 .

For removal of (BOD), (NH₃⁺) and (NO₃⁻) in constructed wetland,

$$C_e = C_i \exp\left(\frac{-A_s K_t y n}{Q}\right) \quad \text{Equation 3.21}$$

For removal of Pathogen in constructed wetland

$$C_e = \frac{C_i}{(1+tK_t)^n} \quad \text{Equation 3.22}$$

3.4.4 Model verification and sensitivity analysis

Model verification was done by varying the input parameters and observing the variations in the output. The inputs that were varied were the average solar radiation, NH_3 -N, BOD₅, and DO, pH, BOD₅ and temperatures. The output that was observed was the simulated FC within the system. The verification was done by varying one parameter at a time (i.e. on varying average solar radiation while keeping the rest of the parameters constant). For the sensitivity analysis, the objective was to see how much the output changed with the inputs. This was carried out by varying the inputs with a relative change of $\pm 10\%$ of the inputs. The sensitivity made it possible to distinguish between high leverage variables, whose values have a significant behaviour and low-leverage variables, whose values have minimal impact on the system (Jørgensen and Bendoricchio, 2001).

4. RESULTS AND DISCUSSION

4.1 Laboratory results and discussion

Laboratory determinations was needed to evaluate the treatment performance of pilot Vertical subsurface flow constructed wetland. It was observed that the river pollution has increased from the previous years.

Common irrigation water quality parameters of the wastewater from Akaki River near German Bride was determined before and after treatment in the pilot vertical subsurface flow constructed wetland (VSSFCW). The laboratory results were compared to WHO standards for wastewater reuse in agricultural irrigation for vegetables likely to be eaten raw. Please see Appendix B.

The selected wastewater quality parameters were BOD₅, COD, TSS, Turbidity, NH_3 , NO_3 , NO_2 , TN, TP, Sulfide, pH, temperature and facial coliform (FC). During the entire period of the study,

four paired (inflow and outflow) samples were analyzed for each parameter.

When the laboratory results for selected parameters from the influent compared to the WHO standards, the wastewater from the river does not fit for irrigation use. However, the concentrations of total phosphorous, nitrate and nitrite are below the standard.

overall statistics of influent and effluent concentrations and removal efficiencies of each parameters, for the first six months of operation i.e., from end of August 2018 until mid-February 2019 are presented in Table 3.

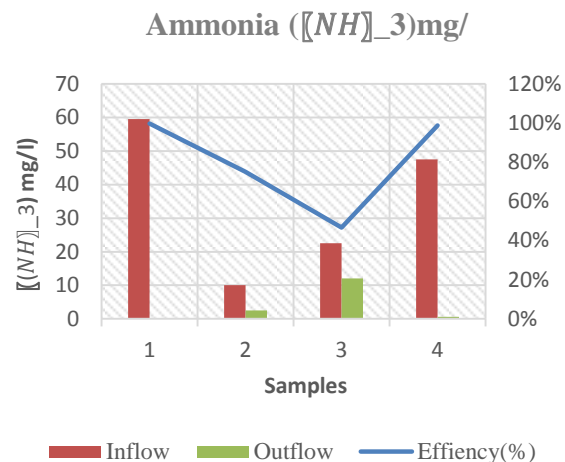
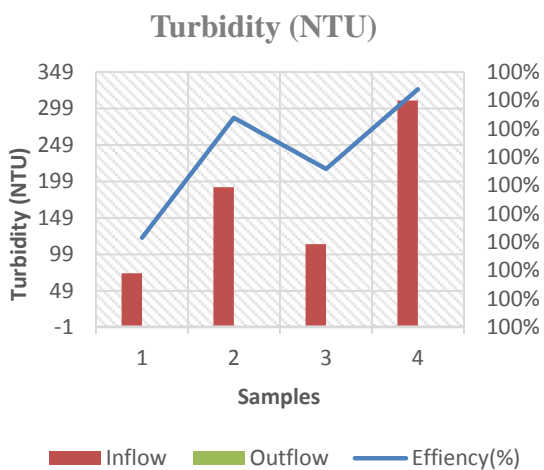
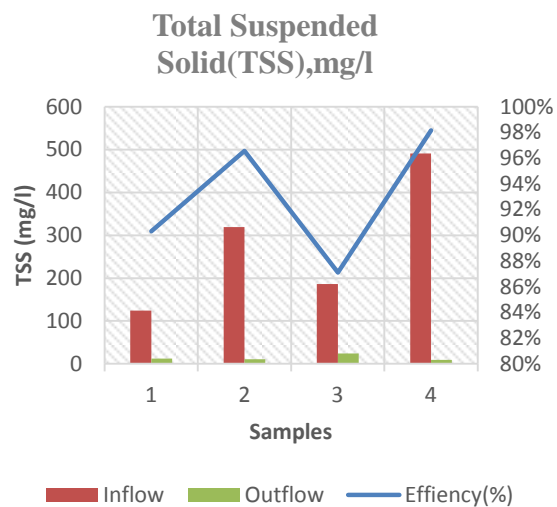
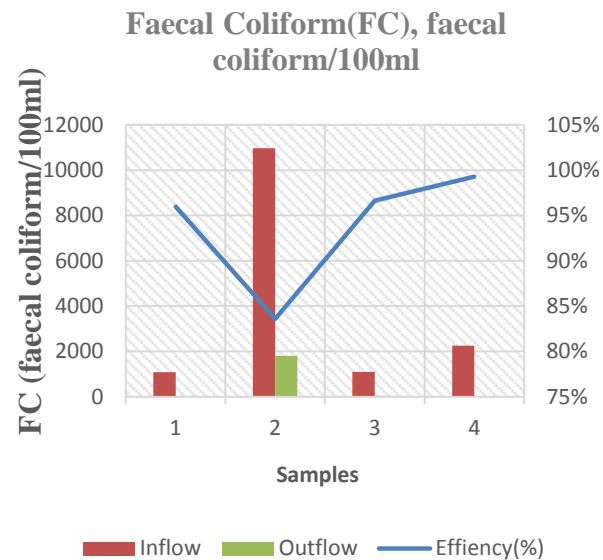
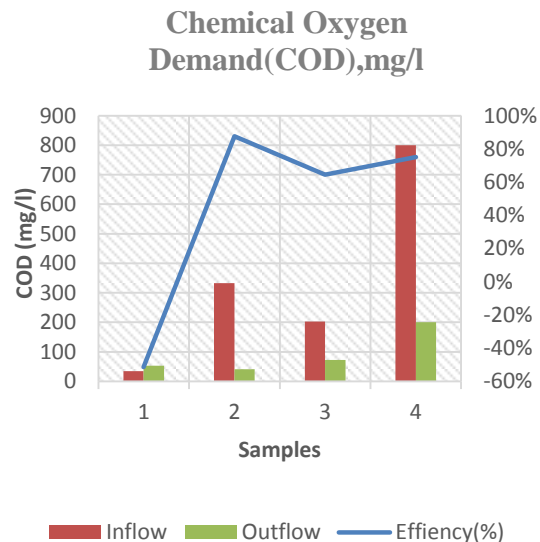
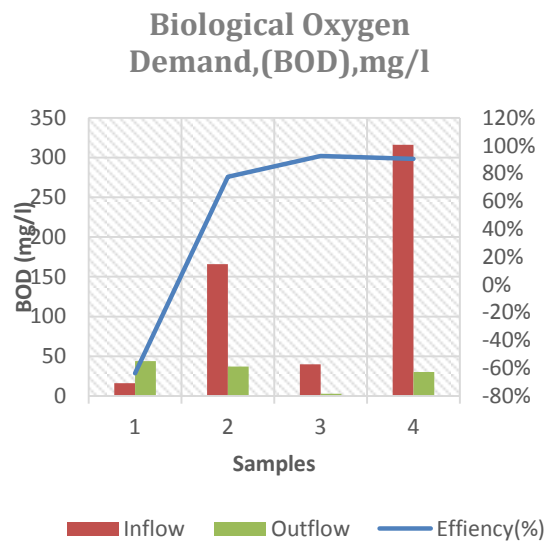
The laboratory results presented in Table 3 are very coarse or varied in a great range as the average \pm standard deviation shows and this was due to the fewer sampling as a result of financial and laboratory limitation.

Table 3. Summary of Influent, effluent concentrations and removal efficiency of pilot scale VSSFCW

Sample No.	BOD (mg/l)			COD (mg/l)			FC (fecal coliform/100ml)			DO (mg/l)		
	In flow	Out flow	R.E (%)	In flow	Out flow	R.E (%)	In flow	Out flow	R.E (%)	In flow	Out flow	
1	16	44	-175	35	53	-51	1090	44	96	8	5.7	
2	166	37	77.7	333	41	88	10975	1800	83.6	4	7	
3	40	3	92.5	203	72	65	1097	37	96.6	5.2	8	
4	316	30	90.5	800	200	75	2254	16	99.3	2	6	
Average	135	28.5	87	342.8	91.5	76	3854	474	94	4.8	6.7	
STDV	137.7	17.9	8	328.3	73.5	12	4779	884	7	2.5	1.0	
(Avg ±SD)	135±13 7.7	28.5 ± 17.9	87 ± 8	342.8 ± 328.3	91.5 ± 73.5	76 ± 12	3854 ± 4779	474 ± 884	94 ± 7	4.8 ±2.5	6.7±1.0	
Sample No.	Turbidity (NTU)			TSS (mg/l)			Ammonia (mg/l)			Nitrite (mg/l)		
	In flow	Out flow	R.E (%)	In flow	Out flow	R.E (%)	In flow	Out flow	R.E (%)	In flow	Out flow	R.E (%)
1	73	<0.0001	100	124	12	90	59.5	0.13	100	0.29	0.03	88.5
2	191	<0.0001	100	319	11	97	10	2.5	75	0.27	0.10	64.8
3	113	<0.0001	100	186	24	87	22.5	12	46.7	0.39	0.01	97.7
4	310	<0.0001	100	491	9	98	47.5	0.6	98.7	0.00	0.21	-5200
Average	171.8	0	100	280.0	14	93	34.9	3.8	80	0.24	0.09	-1237
STDV	104	0	0	162.5	6.8	5.2	22.6	5.6	25	0.16	0.09	2642
(Avg ±SD)	171.8 ± 104	0	100 ± 0	280 ± 162.5	14 ± 6.8	93 ± 5.2	34.9 ± 22.6	3.8 ± 5.6	80 ± 25	0.24 ± 0.16	0.09 ± 0.09	-1237 ± 2642

Sample No	Nitrate (mg/l)			Sulfide (mg/l)			TN (mg/l)			TP (mg/l)		
	In flow	In flow	R.E (%)	In flow	Out flow	R.E (%)	In flow	Out flow	R.E (%)	In flow	Out flow	R.E (%)
1	-	-	-	0.18	0.02	88.8	-	-	-	-	-	-
2	-	-	-	-	-	-	19.4	12.2	37.1	0.98	0.55	43.9
3	3.6	3	16.7	0.33	0.04	87.9	21.1	10.5	50	0.54	0.28	48.1
Average	3.6	3	16.7	0.25	0.03	88.3	20.2	11.4	43.6	0.76	0.42	46
STDV	3.6	3	0	0.11	0.01	1	1.2	1.2	9.1	0.31	0.19	3
(Avg ±SD)	3.6 ± 3.6	3 ± 3	16.7 ± 0	0.25 ± 0.11	0.03 ± 0.01	88.3 ± 1	20.2 ± 1.2	11.4 ± 1.2	43.6 ± 9.1	0.76 ± 0.31	0.42 ± 0.19	46 ± 3

Sample No	Temperature (°C)		PH		Sampling dates	
	In flow	Out flow	In flow	Out flow	Sample No.	Sampling date
1	19	18	7.3	7.5	1	August
2	25.5	23	7.7	8.8	2	September -October
3	24.5	20.5	7.9	8.2	3	December - January
4	22.1	22.3	7.4	8.3	4	February
Average	23	21.0	8	8	-	-
STDV	3.5	2.2	0.3	1	-	-
(Avg ±SD)	23 ± 3.5	21 ± 2.2	8 ± 0.3	8 ± 1	-	No sampling carried out



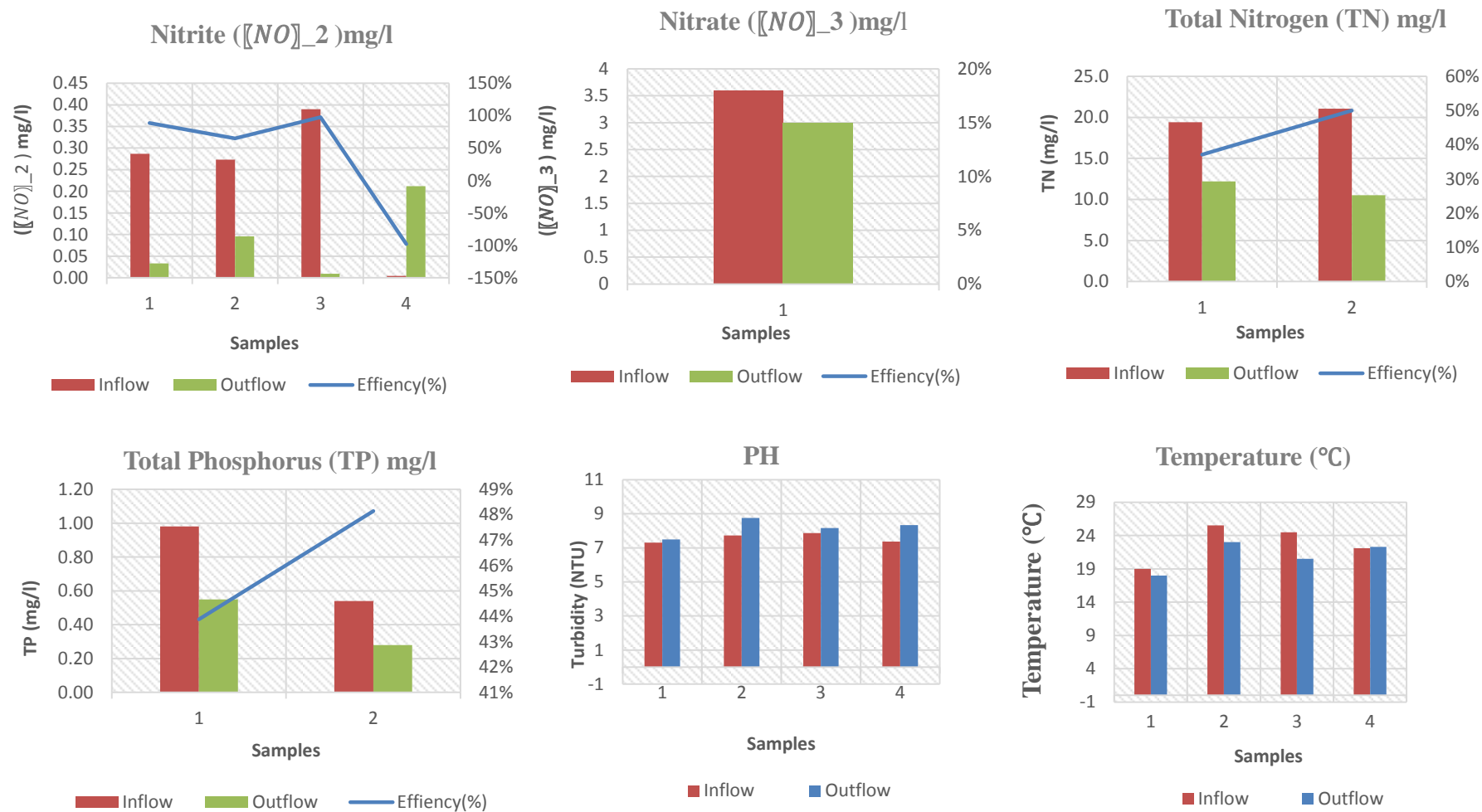


Figure 14 Average removal efficiency of VSSFCW at different sampling times

4.1.1 Physicochemical and biological removal

The pollutant load of the influent wastewater from the river varied widely throughout the experiment; and this makes the effluent from the pilot VSSFCW to show a related variation. This was clearly seen from the graphs on figure 10 for BOD, COD, TSS and turbidity. The graphs show that the changes in the influent and effluent concentration for BOD, COD, TSS and turbidity throughout the course of the experiment were variable, and that indicates the ability of the wetland to work under different wastewater strength influents. However, when the laboratory results of effluent concentration compared to WHO standards, BOD, COD and faecal coliform have exceeded the guideline. Table 4 shows the comparison of average effluent concentration with WHO standard.

Table 4. Comparison of pilot plant VSSFCW average effluent concentration of physicochemical and biological parameters with WHO (2006) standard

Water quality parameter	BOD (mg/l)	COD (mg/l)	FC (per 100 mL)	Turbidity (NTU)	TSS (mg/l)
Current study effluent	28 ± 21	91.5±73.5	(627±1015)	<0.0001	15.7 ± 7.2
WHO (2006) standard	20	100	200	*	15

* No specific standards exist for wastewater reuse.

The first laboratory result shows an increased BOD and COD concentrations towards the outlet. This resulted in negative removal efficiency of the pilot VSSFCW. The influent concentration of BOD for the first sample was 16mg/l. However, the effluent was 44mg/l. The low BOD influent level was due to the high rainfall during August and the river was diluted and oxygenated by the rain. However, when this wastewater goes through treatment in the pilot VFSSCW the BOD increases. This is due to the microorganisms in the waste and their increasing demand of oxygen for biodegradation.

Debabrata (2013) and Knight (1999) related this phenomena with the effect of background

concentration (C^*) of the wetland. Most of the wetlands have a background BOD, which is induced from the partial decomposition of previously settled influent solids, plant, and additional by-products from anaerobic digestion. The background concentration of wetlands is well discussed in Chapter 2 Section 2.13.5.

Debabrata, (2013) further explains that the reduction in BOD_5 are significant for high incoming concentrations but less when the inlet concentrations near background concentration. (C^*). The function of wetlands in removing BOD is envisaged only when the incoming BOD is higher than the natural background level.

The wetland background concentration for BOD_5 typically ranges from 3 to 15mg/ depending on the strength of the wetland carbon cycle (Knight, 1999). The inflow BOD background concentration for this research was calculated to be 4.3 mg/l. However, for higher BOD loadings, for example 316 mg/l, the background concentration was 20.2 mg/l. Hence, the lower inflow concentration more approached to the background concentration than the higher inflow concentrations.

However, this situation reduced when the treatment plant reaches its full potential and when the plants grow fully to bring oxygen to the treatment plant. Moreover, effluent aeration by making the effluent to fall from raised outlet structure can increase dissolved oxygen in the effluent. As a result, the negative removal efficiency of the pilot VSSFCWW showed a faster recovery capability to the acceptable removal efficiency.

Since COD values are always higher than BOD values, the influent from the river also had unpredictable amount of COD (342.8 ± 328.3 mg/l) and BOD (135 ± 137.7 mg/l). The magnitude of the decrease in pollutant concentration upon treatment also had a varied range i.e., for COD (91.5 ± 73.5 mg/l) and for BOD (28.5 ± 17.9 mg/l). Pilot VSSFCW effluent concentration was compared with the provisional discharge limit values set by WHO. Although, lower concentrations of pollutants were measured in the individual sampling, the mean effluent is slightly higher than the standard concentration recommended by WHO. For example, the individual COD effluent concentration exceeds this standard value only once, i.e., when the

influent value reaches 800mg/l, the effluent was 200mg/l. However, the effluent levels of BOD is high except one occasion when the influent was 40mg/l and effluent becomes 3mg/l.

VSSFCW effluent concentration values of TSS (15.7 ± 7.2 mg/l) and Turbidity (<0.0001 NTU) do not display any measurable value in the outlet. This indicates the effectiveness of the constructed wetland in fulfilling the regulatory limit values to reuse the effluent for vegetable irrigation.

The mean effluent values for Fecal coliform bacteria (474 ± 884 fecal coliform/100ml) were much higher than the standard concentration recommended by WHO guidelines of effluent reuse aimed at agricultural irrigation for vegetables likely to be eaten raw. This higher effluent (1800 fecal coliform/100ml) load was gained to the higher influent load (10975 fecal coliform/100ml). However, except this higher load, VSSFCW effluent levels (44, 37, 16 fecal coliform/100ml) for other influent loads (1090, 1097, 2254 fecal coliform/100ml respectively) were much lower than the standard limit. Results in Table 3 indicated that only for higher influent loads that the VSSFCW showed relatively higher effluent concentrations.

The pilot VSSFCW shows effective average removal efficiency on Physicochemical and biological parameters from the waste water. The result in table 3 indicated that the average removal efficiencies of chemical oxygen demand (COD), biological oxygen demand (BOD), total suspended solids (TSS), Turbidity and Fecal Coliform (FC) were 76%, 87%, 93%, 100%, and 94% respectively. Thus, after well ripening period the pilot treatment plant continues to give an improved and stable result.

The results of this study are consistent with other studies. Sara G et al, (2016), for example, reported similar finding with COD, BOD, TSS, Turbidity and FC removal efficiency of VSSFCW. Table 5 presented results of the present study in reference to Sara G et al, (2016) and Akaki WWTP (2007).

Table 5. Pollutant removal efficiency comparisons with other study

Data source	Water quality parameter	BOD (mg/l)	COD (mg/l)	TSS (mg/l)
Current study	In flow	135	342.8	280
	Out flow	28.5	91.5	14
	Removal efficiency	87 %	76 %	93 %
Akaki WWTP(2007)	In flow	168	286	512
	Out flow	14	66	54
	Removal efficiency	77%	92%	89%
Sara G et al, (2016) VSSFCW	In flow	253	465	227
	Out flow	38	110	45
	Removal efficiency	84 %	76 %	80 %

The treatment efficiency of the pilot VSSFCW shows comparable performance in BOD and COD for slightly different influent concentrations. For TSS removal when compared to the Akaki WWTP (2007) there was large influent variation but the VSFCW show good efficiency as the Akaki WWTP. Better efficiency in TSS removal was seen than Sara G et al, (2016). However for BOD, & COD there was comparable removal efficiency in the VSSFCW with Sara G et al, (2016), but there was difference in the influents concentrations to the treatment plants.

The high removal efficiency of TSS might be due to the fact that, in SSF constructed wetlands the water flows below the ground through gravel and wetland plant roots. This facilitates the physical, chemical and biological wastewater treatment mechanisms such as sedimentation, aggregation, surface adhesion and biodegradation (Mergaert et al., 1992). These processes improve the efficiency of removing TSS in the SSF constructed wetland within a short hydraulic retention time, of 2 to 5 days (USEPA, 1993).

4.1.2 Nutrients removal

To analyze the nutrient removal in pilot VFSSCW, the influent and effluent ammonia (NH_4), nitrate (NO_3), nitrite (NO_2), total nitrogen (TN) and total phosphorus (TP) were evaluated. Table

3 also shows the mean influent and effluent concentrations of nutrients from pilot VFSSCW. The mean influent parameter values were NH_3 (34.9 ± 22.6 mg/L), NO_3 (3.6 ± 3.6 mg/L), NO_2 (0.24 ± 0.16 mg/L), TN (20.2 ± 1.2 mg/L), and TP (0.76 ± 0.31 mg/L). The effluent results for these influent concentrations then compared against WHO(2006) standards for wastewater effluent reuse for irrigation of vegetables likely to be eaten raw are presented in Table 6.

Table 6. Comparison of pilot plant VSSFCW, average effluent concentration of Nutrients with WHO (2006) standard

Water quality parameter	NH_3 (mg/l)	NO_3 (mg/l)	NO_2 (mg/l)	TN(mg/l)	TP (mg/l)
Current study effluent	3.8 ± 5.6	3±3	0.09 ± 0.09	11.4 ± 12	0.42 ± 0.19
WHO (2006) standard	5	30	*	15	2

*No specific standards exist for wastewater reuse.

During this research nitrate (NO_3), nitrite (NO_2) and Total Phosphorus influent concentrations were below the standard limit of WHO. However, the river water quality concentrations are highly variable between sampling periods; therefore, there may be time for higher concentrations of these parameters. Therefore, this makes necessary to study the efficiency of the pilot vertical subsurface flow constructed wetland for (NO_3), (NO_2) and TP.

Better removal efficiency obtained in comparison to Chen et al (2007), which is a study on the efficiency of vertical subsurface-flow constructed wetland on wastewater from Longdao River in Beijing China.

Table 7. Pollutant removal efficiency comparisons with other study

Water quality parameter	NH_3 (mg/l)	TN (mg/l)	TP (mg/l)
Current study	80%	44 %	46 %
(Chen et al, 2007) VSSFCW	77.4 %	10.6 %	30.6 %

Total Nitrogen typically consists of varying proportion of particulate organic Nitrogen, dissolved organic Nitrogen, Ammonium Nitrogen, Nitrite Nitrogen and Nitrate Nitrogen (Reddy and

Patrick, 1984; Kadlec and Knight, 1996). Total Nitrogen removal was not high due to poor Nitrate removal in the pilot treatment plant (Valerie. J, 2009). In subsurface constructed wetland system, mineralization transforms these organic Nitrogens to its inorganic constituents. Because of the optimum PH value 8 ± 1 , in the pilot plant VFSSCW treatment, the hydrolysis of organic Nitrogen resulted mainly to Ammonium-N. This pathway occurs under both aerobic and anaerobic conditions and it is often referred to as ammonification (USEPA, 1993).

Nitrification limits nitrogen removal in many CWs because ammonia is the dominant species of nitrogen in urban and other industrial wastewaters. The degree of nitrification is determined by oxygen availability (Cottingham et al., 1999). Therefore, vertical SSF CWs are more conducive to nitrification than horizontal SSF CWs. The aerobic condition in the superficial wetland bed and anaerobic condition in the bottom allow the nitrogen removal in the wetland.

The better ammonia removal in gravel beds is most likely an effect of greater surface area for attached-growth nitrifying bacteria and higher oxygen, as discussed in chapter 2 section 2.10.4, since nitrifying bacteria are aerobic (Gray 2004). Furthermore, the assumed longer HRT of the wastewater in gravel beds gives more time for nitrification to take place, which is in agreement with Cui et al. (2010) who found that NH_3^+ and TN were removed more efficiently in VF beds with a longer HRT.

The second ammonium pathway is biological uptake. Unlike most terrestrial plants, many aquatic plants use ammonium as a nitrogen source (Kadlec & Knight, 1996; Simi & Mitchell, 1999; Brix et al., 2003).

Accordingly, the pilot treatment plant of VFSSCW shows good nitrification efficiency result. This indicates there is enough oxygen in the wetland creating aerobic zone for nitrification. Also, to increase this zone, some inspection pipes are used to allow oxygen to the inside of the pilot treatment plant.

Nitrification itself cannot remove nitrogen from wastewaters, but nitrification coupled with denitrification seems to be the major removal pathway in many CWs (Mayo & Bigambo, 2005;

Tanner et al., 2002; Vymazal, 2007).

Denitrification is a process in which nitrite and nitrate are mainly converted into harmless nitrogen gas that escapes from the wetland. The optimum pH level for denitrification is between pH 6 and 8 . Denitrification is also strongly dependent on temperature most likely above 5 degree centigrade. These both criteria on the pilot treatment plant was satisfied with PH (8 ± 1) and temperature is (21.5 ± 2.2). However, denitrification occurs slowly at the pilot treatment plant, but still to a significant degree. The small transformation of nitrate to nitrogen gas might be due to the low anaerobic zone for denitrification process at the bottom zone of the pilot plant VFSSCW.

Progressive accumulation of organic matter in the granular medium of SSF CWs increases denitrification rates. Organic wastes such as bark or straw can be mixed with the granular medium in wetlands to increase denitrification rates, particularly at the beginning of the operating period, when the systems contain only a small amount of accumulated organic matter (Sovik & Morkved, 2008).

Incomplete denitrification leads to the release of nitrous oxide, which is a strong greenhouse gas. Teiter and Mander (2005) found that vertical SSF CWs emitted a considerably higher volume of nitrous oxide than horizontal SSF CWs. Nevertheless, although nitrous oxide emissions were found to be relatively high, their overall influence on climate change was estimated to be insignificant.

The phosphate concentration in the treated wastewater ranged between 0.2 mg/l and 0.5mg/l with an average percentage removal of 46%. This result is an intermediate among those recorded in other studies carried out using VFCW. P-removal rates in vertical systems seem to be more variable than N removal rates, and different authors have reported average concentration based total phosphorus (TP) removal efficiencies of 25–40% (Verhoeven & Meuleman, 1999) and 26–70% (Rousseau et al, 2004). In all of these studies, TP effluent concentrations were above 2 mg/l and increased over time. The moderate removal rate of phosphorus in this study (VSSFCW) may be attributed to the long contact time with in the wetland (8 days) and the use of three different

plants in the same basin, which increase the uptake of phosphorus (Klomjek & Nitorisavut, 2005).

Three main factors lead to low P-removal rates in wetlands. First, microbial phosphorus removal is only a temporary sink. Second, as in the case of nitrogen, plant uptake tends to have a relatively small effect on removal. Third, most substrates used as granular media have low P-sorption and P-complexation capacities: in the case of gravel, from the start of the operating period; in the case of sandy media, after a limited period of time due to the depletion of sorption sites and complexing agents.

On the other hand, since the final effluent is mainly used for irrigation purposes the low removal efficiencies on Total Phosphorous and nitrogen is the desired one for the vegetables.

3.2 Model results and discussions

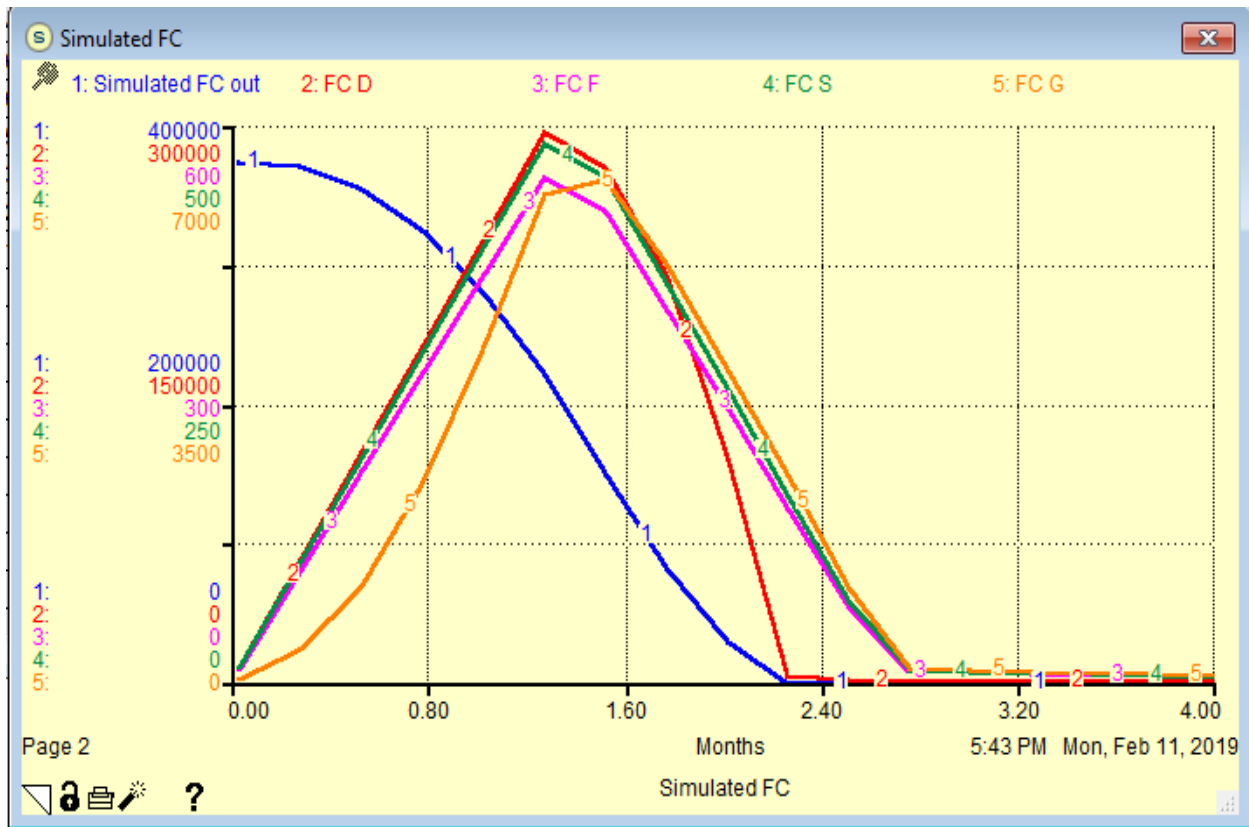


Figure 15. STELLA graphical representation of Faecal Coliform Removal in VSSFCW

4.2.1 Removal mechanisms

The removal of FC bacteria were followed by the combined effects; natural die off, adsorption, filtration and sedimentation. From the graph in the figure 15 it can be seen that each mechanisms have different magnitudes on the removal process at different sampling periods. Moreover table 8 presents' model average results for each removal mechanisms which are taken from the model output data table on figure 15.

Table 8. Average removal magnitudes of different processes

Faecal counts /100ml	Simulated FC out	FC _f	FC _s	FC _d	FC _{ip}	FC _{op}	FC _g	Observed FC out
Average FC	129303.7	189.21	168.64	90829.15	994.73	90.19	2047.17	585.31
STDEV FC	151250	190.28	169.60	108306.1	703.18	116.95	2240.22	589.10
AV±SD	129303.7± 151250	189.21 ± 190.28	168.64± 169.60	90829.15± 108306.1	994.73± 703.18	90.19± 116.95	2047.17± 2240.22	585.31± 589.10

From the above table it can be seen that the simulated FC out and the observed FC out differs widely. The reason behind these values is due to the initial input value given to the simulated FC out which is three hundred fifty thousand FC counts per 100ml, in order to be used throughout the computational process by different faecal coliform removal processes in the wetland system.

Die-off process

It was observed that die-off process was the major removal mechanism in the pilot vertical subsurface flow constructed wetland. The average result in table shows that about 90829.15 ± 108306.1 FC Counts per 100ml are removed through the die-off process. The reasons behind might be unfavorable environmental conditions to microorganisms which were PH, temperature and solar radiations.

Predation by other organisms is also plays a role in the die off process of faecal coliforms. Kadlec & Knight (1996) showed that vegetated wetlands seem more effective in pathogen removal since they allow a variety of microorganisms to grow which may be predators to pathogens.

Filtration process

The second process that contributed much in the removal of FC was filtration. In this process there were 189.21 ± 190.28 FC counts per 100ml per day. It was supposed that the suspended solids in the wastewater that are filtered within the system contained some faecal coliform bacteria.

Sedimentation process

The sedimentation process that contributed less in the removal of FC with 168.64 ± 169.60 FC Counts per 100ml per day. The FC in sedimentation process might be due to the settling organic matters which suspected to settle with the FC bacteria. This process was governed by the sedimentation rate in the VSSFCW.

Growth processes

Growth process on the other hand increases the FC concentration within the system. It was observed that the growth process had 2047.17 ± 2240.22 FC Counts per 100ml per day. This is due to the bacterial growth attached to the gravel and plant roots in the VSSFCW system. Attached growth was the main treatment mechanism in subsurface constructed wetlands. Attached microorganisms are responsible to digest organic matter in the wastewater both aerobically and anaerobically. This process plays a great role in reduction of organic matter and nutrients in wastewater.

4.2.3 Verification and sensitivity analysis results

A sensitivity analysis with 10% relative parameter changes revealed that the hydraulic parameters were the most influential, followed by the oxygen release rate, yield coefficients, and lysis rates for the bacteria. This changes are done by increasing and decreasing the input parameter by the following percentages 10%, 9%, 8%, 7%, 6%, 5%, 0%, -5%, -6%, -7%, -8%, -9%, -10%. This relative changes are represented on the simulation graph by numbers from 1 to 13 respectively. As a result the STELLA model runs the simulation 13 times.

Solar radiations showed to have an effect of decreasing the FC in the simulated model please see From figure 14. As the solar radiation increases there is a decrease of simulated FC within the system. At a decreasing rate of solar radiation from 10% to -10% of the relative change, the simulated FC were increased. This shows that solar radiation have an inverse relation on faecal coliform removal. The simulation input and output table is presented in appendix I.

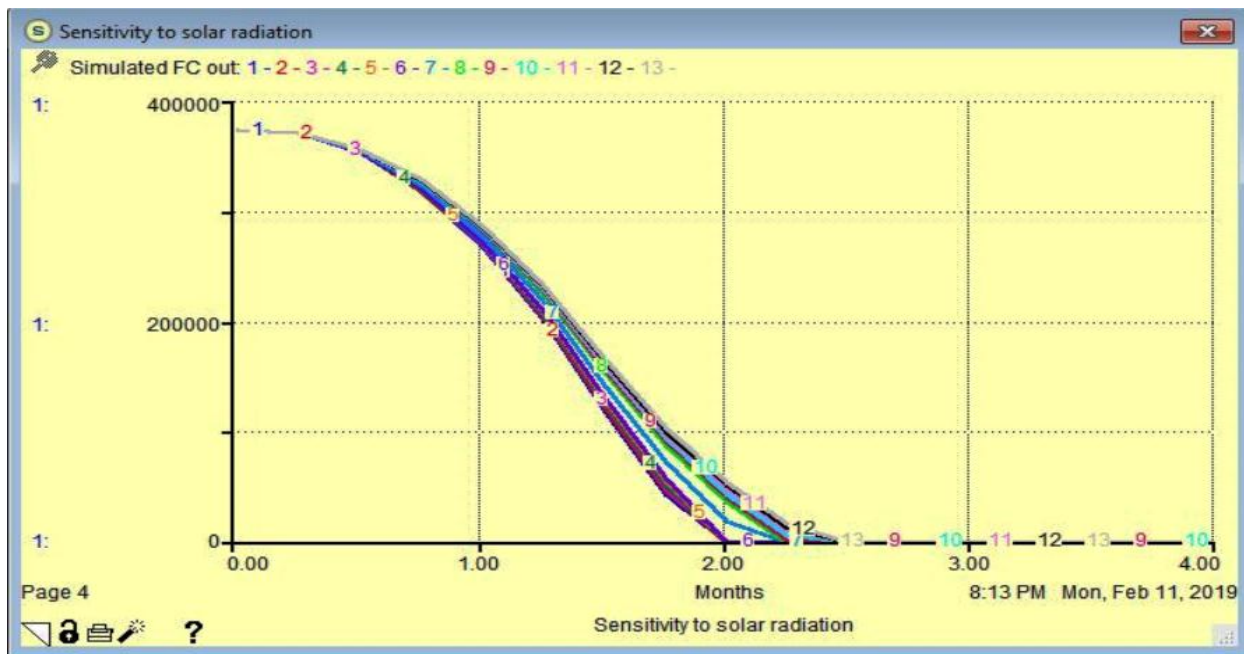


Figure 16. Simulated FC to solar radiation changes

The graphs from figures 15 and 16 showed that temperature, PH, BOD, and Ammonia showed to have no effects to the death of FC bacteria in this model. Then, it can be concluded that the effect of this parameters in the input to the output was insignificant.

Figure 17. Sensitivity of simulated FC to Ammonia and BOD

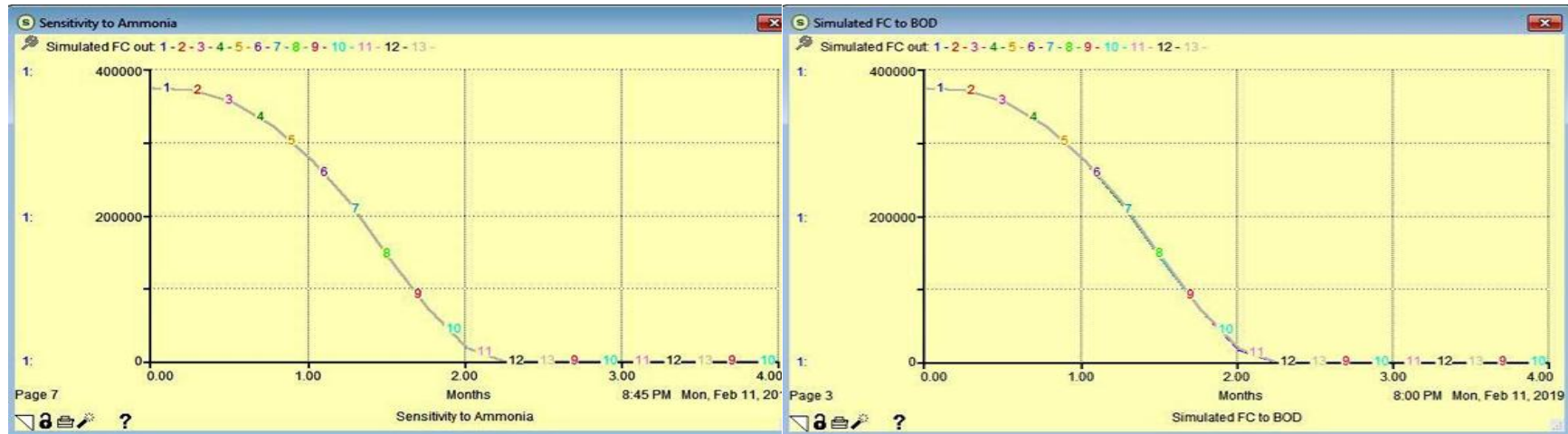
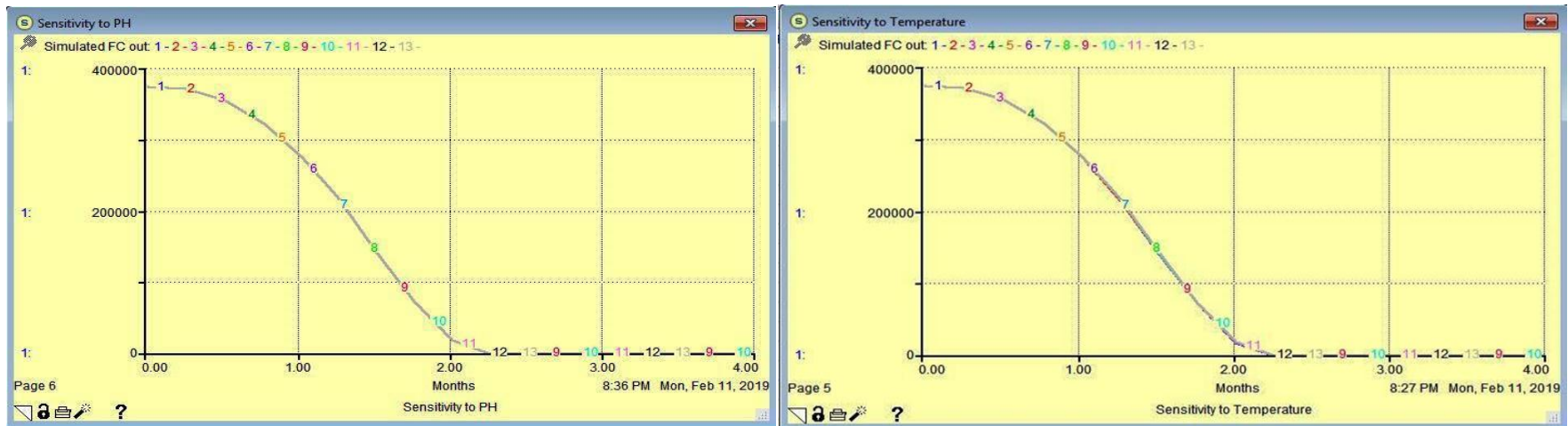


Figure 18. Sensitivity of simulated FC to PH and Temperature



4.2.4 Model calibration

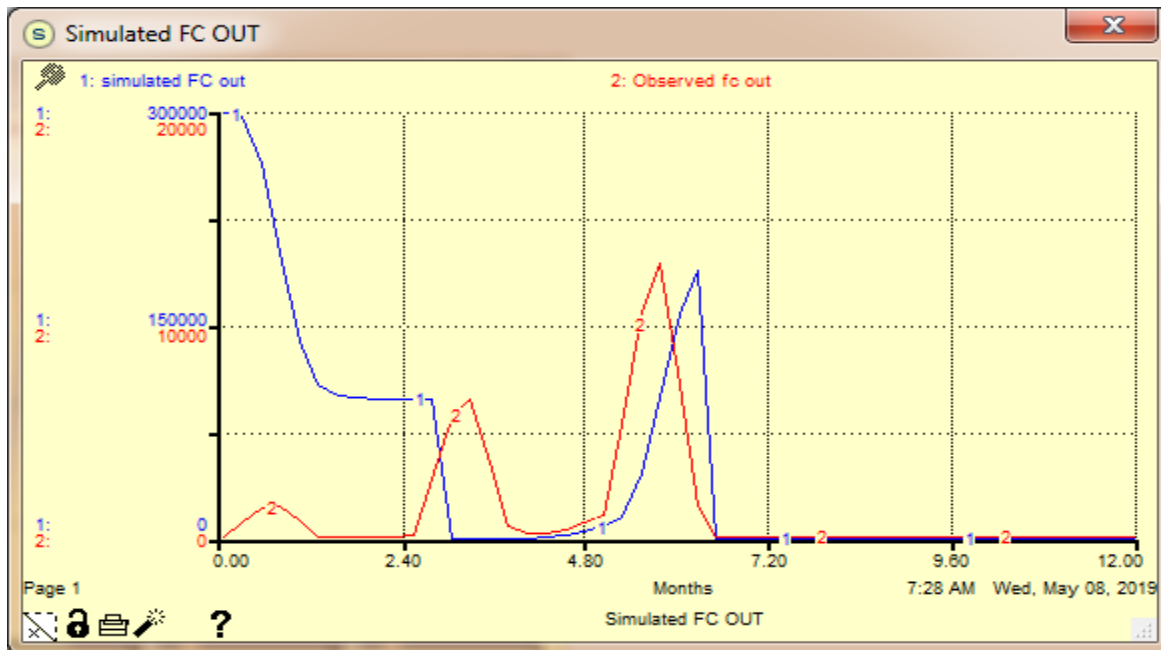


Figure 19. The simulated and observed results of FC removal in VSSFCW

The parameters and constants are presented in appendix II. The calibration results showed that simulated Faecal Coliform followed the trend of observed Faecal Coliform; however the shift of the simulated value from the observed value is due to the initial value which is given to the model for simulation FC out. This initial value given to the model is three thousand FC counts per 100ml. At first the simulated FC out is above the observed value since initially the simulated value is higher than the higher values of the observed values and finally the simulated value runs following the trends of the observed FC out. This tells that the developed model was successfully run.

The major kinetic coefficients that were used in this study in model calibration were the light mortality constant and temperature coefficient.

However other kinetic coefficients used in this study, which are the Half Saturation Constants of BOD, DO, NH_3 and pH, sedimentation rate and filtration rate were kept as they were in the literatures. It was seen that they have no significant effect in calibration of the model. Please see table 9.

Table 9. Calibrated values of the simulated FC model

Symbols	Units	Literature	Calibrated
ΦT (or OT)No.		1.04	0.00009
Φ (or OL)	m^2/cal	0.035	0.0000551
I_{avg}	$cal/m^2.d$	4558-6450	6100
r_{20}	d^{-1}	0.05-1.14	1
sr	m/d	0.0864	57

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

- Vertical subsurface flow constructed wetland (VSSFCW) system has higher treatment efficiency for the biological and physicochemical parameters and satisfactory efficiency for nutrient removal, from non-point source polluted Akaki River so as non-point source polluted wastewater.
- The biological characters of treated wastewater were slightly above with the WHO standard for wastewater reuse in Irrigation of crops likely to be eaten uncooked. However the effluent can be used for restricted irrigation of vegetables. No odours or insects were detected during the operation of pilot vertical subsurface flow constructed wetland, beside a beautiful landscape view.
- The presence of diverse plant species in VSSFCW provided a more effective distribution of the root system and habitat for more diverse microbial populations and consequently better removal rates.
- The developed model was a success since it demonstrated its effectiveness in optimising the FC removal in VSSFCW and the die-off process was the best removal mechanism for Faecal coliform removal in VSSFCW treatment system.
- Generally, one can conclude that the treatment performance of VSSFCW wetland system was very encouraging in promoting the use of constructed wetlands as an alternative wastewater treatment system for improving urban agriculture practice along Akaki river side with untreated effluents. In addition, for developing countries like Ethiopia, that have limited resources for the construction and operation of conventional treatment plants, constructed wetlands are the most economical solutions.

5.2 Recommendation for further work

- Vertical subsurface flow constructed wetlands should be used by farmers on site to improve the increased urban agriculture practice in Akaki River catchment.
- For further work it can be recommended to explore heavy metal removal efficiency of the VSSFCW on the polluted Akaki river wastewater, as a case study.
- From the current study, it is evident that the future direction of CW modelling work should be focused to quantify the rates of individual processes which are happening inside various mechanisms are vital because it provides valid information to the designers for the efficient design of CWs. Once such kind of information is available, design recommendation can be made for sizing and the pollutant removal can be estimated in great detail. However, process based models for the removal of pollutants in the CWs is still in its infant stage in Ethiopia and more technical and scientific study is required to improve the understanding of these complex processes.
- From this study it was seen that plants under investigation can survive and well develop under the climatic conditions of Ethiopia. So it can be recommended that in Addis Ababa river side greenery and buffer zone protection projects the microphyts (*C.papyrus*, Cattail and Bulrussh) can be used for both treatment of the river inflow and landscaping.

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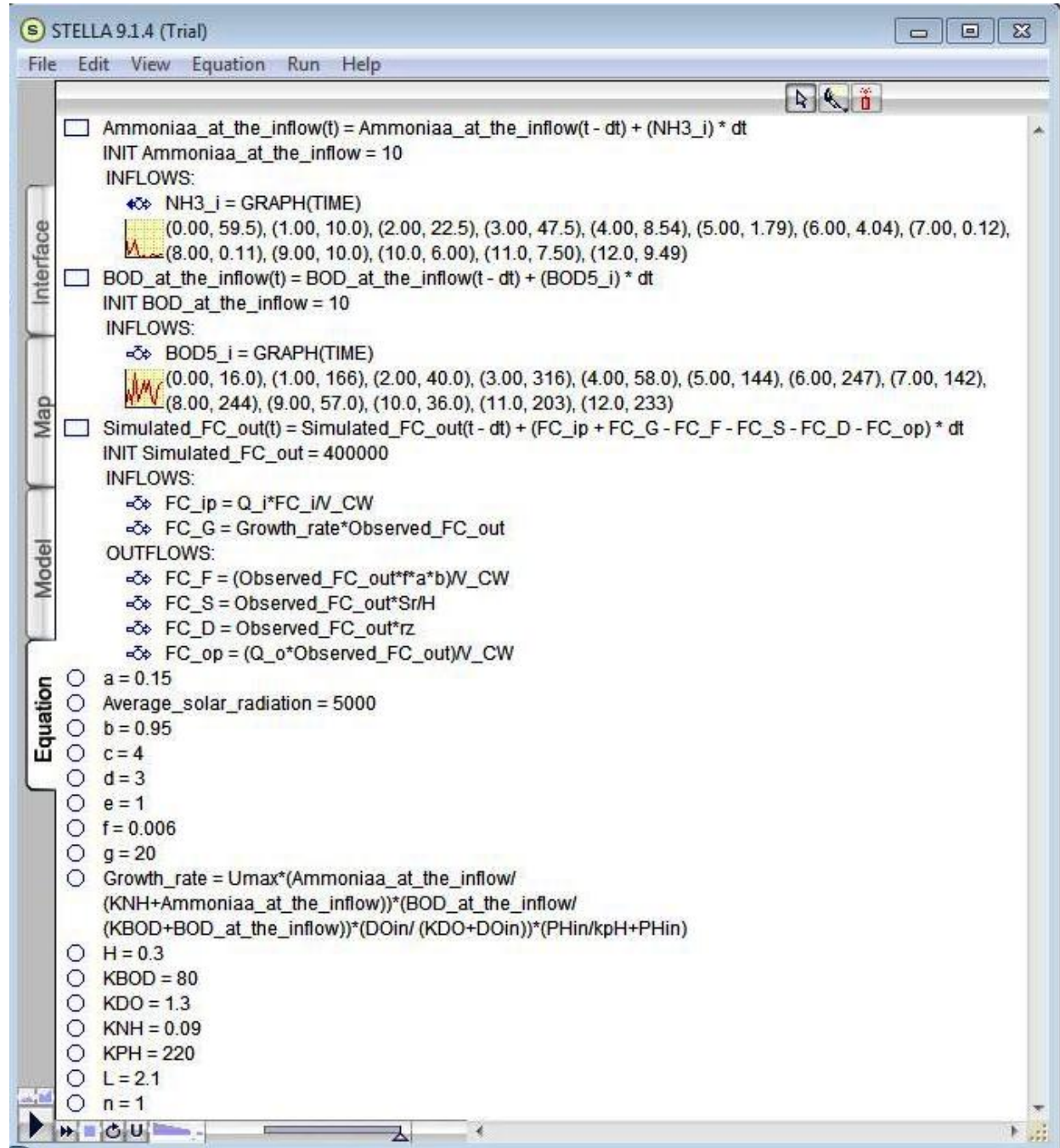
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APPENDIX I

Figure 20. Model input data



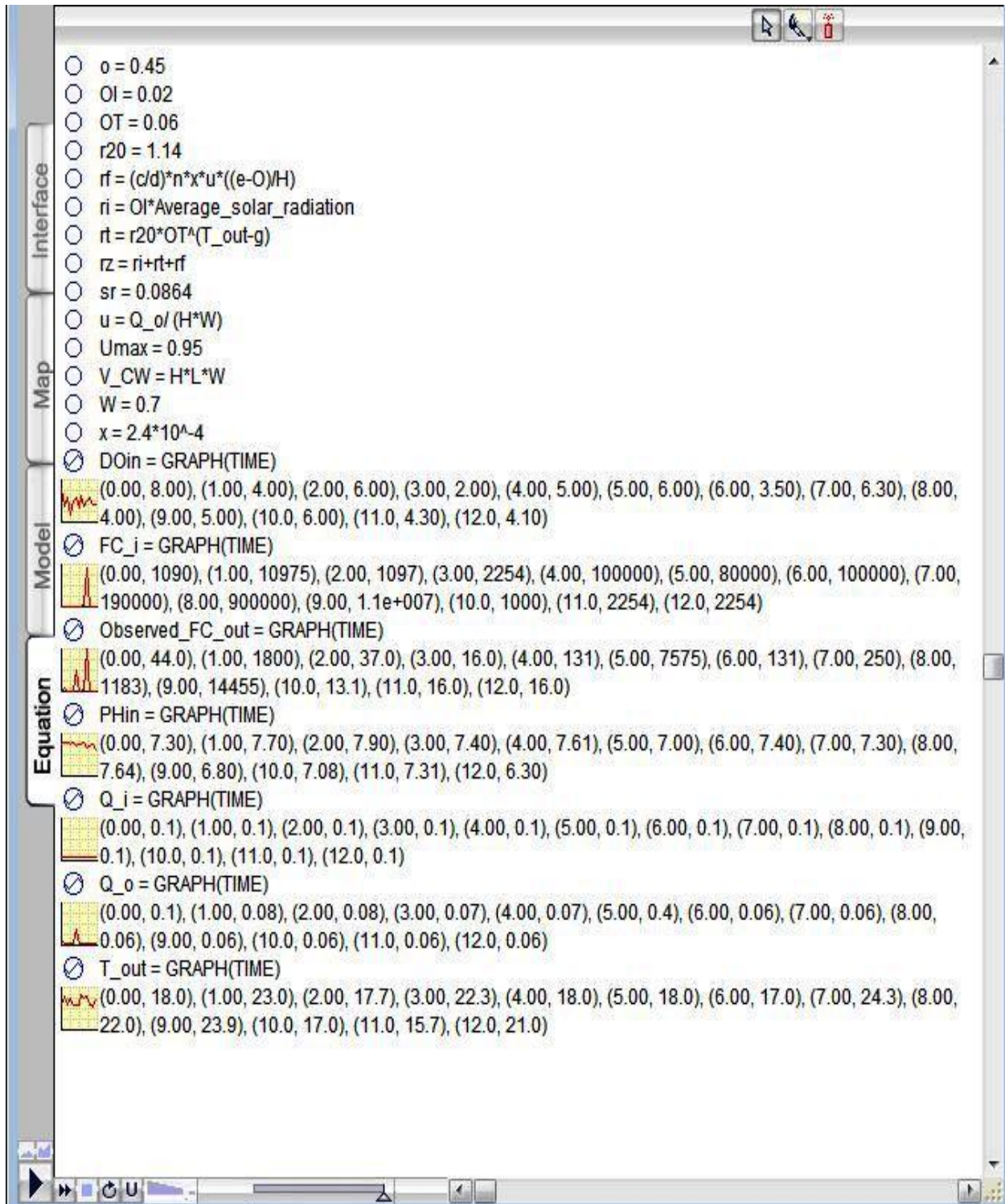


Figure 21. Model output to simulated faecal coliform

sample number	Simulated FC	FC F	FC S	FC D	FC G	FC ip	FC op	Observed FC
.00	375,000.00	14.22	12.67	7,746.39	34.24	247.17	9.98	44.00
.25	373,124.54	120.61	107.50	65,726.99	407.20	667.45	81.46	373.25
.50	366,884.06	227.00	202.32	123,734.61	1,207.73	1,087.73	147.35	702.50
.75	326,380.11	333.39	297.14	181,770.79	2,527.89	1,508.01	207.64	1,031.75
1.00	281,736.84	439.78	391.97	239,837.09	4,239.43	1,928.29	262.32	1,361.00
1.25	223,045.98	546.17	486.79	297,935.21	6,164.84	2,348.57	311.41	1,690.25
1.50	150,354.43	510.42	454.93	278,435.68	6,363.41	2,208.67	286.55	1,579.63
1.75	82,575.56	403.61	359.73	220,139.61	5,283.30	1,788.69	226.59	1,249.06
2.00	29,061.17	296.79	264.53	121,004.01	3,951.94	1,368.71	0.00	918.50
2.25	0.00	189.98	169.33	3,235.30	2,645.88	948.72	0.00	587.94
2.50	0.00	83.17	74.12	1,570.57	1,199.12	528.74	0.00	257.38
2.75	0.00	11.53	10.28	413.28	169.94	265.15	0.00	35.69
3.00	0.00	10.26	9.14	447.48	152.54	314.34	0.00	31.75
3.25	0.00	8.99	8.01	482.05	135.51	363.53	0.00	27.81
3.50	0.00	7.71	6.88	516.39	118.25	412.73	0.00	23.88
3.75	0.00	6.44	5.74	550.09	100.36	461.92	0.00	19.94
Final	0.00							16.00

Figure 22. Model output to sensitivity analysis

Sensitivity to Temperature													
Table 4 (Sensitivity to Temperature)													
Months	1: Simulated	2: Simulated	3: Simulated	4: Simulated	5: Simulated	6: Simulated	7: Simulated	8: Simulated	9: Simulated	10: Simulated	11: Simulated	12: Simulated	13: Simulated
.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00
.25	373,064.95	373,064.77	373,064.59	373,064.41	373,064.21	373,064.02	373,062.94	373,061.69	373,061.41	373,061.13	373,060.83	373,060.53	373,060.22
.50	356,311.75	356,310.68	356,309.58	356,308.46	356,307.31	356,306.15	356,299.91	356,292.94	356,291.45	356,289.92	356,288.36	356,286.75	356,285.11
.75	324,826.65	324,825.35	324,824.03	324,822.68	324,821.30	324,819.90	324,812.46	324,804.21	324,802.45	324,800.65	324,798.81	324,796.93	324,795.01
1.00	278,714.50	278,715.62	278,716.72	278,717.81	278,718.87	278,719.91	278,724.74	278,728.88	278,729.62	278,730.32	278,730.98	278,731.61	278,732.20
1.25	218,042.83	218,051.73	218,060.57	218,069.33	218,078.01	218,086.62	218,128.48	218,168.29	218,175.99	218,183.60	218,191.13	218,198.55	218,205.89
1.50	142,828.67	142,854.34	142,879.68	142,904.69	142,929.37	142,953.73	143,070.58	143,179.32	143,200.10	143,220.55	143,240.69	143,260.50	143,279.99
1.75	72,692.28	72,733.62	72,774.39	72,814.59	72,854.22	72,893.29	73,080.23	73,253.39	73,288.38	73,318.84	73,350.76	73,382.15	73,413.00
2.00	17,337.75	17,388.72	17,439.00	17,488.61	17,537.55	17,585.82	17,817.16	18,032.09	18,073.13	18,113.53	18,153.28	18,192.40	18,230.88
2.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Final	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Sensitivity to PH													
Table 5 (Sensitivity to PH)													
Months	1: Simulated	2: Simulated	3: Simulated	4: Simulated	5: Simulated	6: Simulated	7: Simulated	8: Simulated	9: Simulated	10: Simulated	11: Simulated	12: Simulated	13: Simulated
.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00
.25	373,062.94	373,062.94	373,062.94	373,062.94	373,062.94	373,062.94	373,062.94	373,062.94	373,062.94	373,062.94	373,062.94	373,062.94	373,062.94
.50	356,299.87	356,299.87	356,299.88	356,299.88	356,299.88	356,299.88	356,299.91	356,299.94	356,299.94	356,299.95	356,299.96	356,299.96	356,299.97
.75	324,812.29	324,812.30	324,812.32	324,812.33	324,812.35	324,812.37	324,812.46	324,812.55	324,812.57	324,812.60	324,812.62	324,812.64	324,812.66
1.00	278,724.31	278,724.35	278,724.39	278,724.43	278,724.47	278,724.51	278,724.74	278,724.99	278,725.04	278,725.10	278,725.15	278,725.21	278,725.27
1.25	218,127.81	218,127.89	218,127.77	218,127.86	218,127.94	218,128.03	218,128.48	218,128.98	218,129.09	218,129.20	218,129.31	218,129.42	218,129.54
1.50	143,069.08	143,069.22	143,069.36	143,069.50	143,069.65	143,069.80	143,070.58	143,071.45	143,071.64	143,071.83	143,072.02	143,072.22	143,072.42
1.75	73,078.08	73,078.28	73,078.48	73,078.68	73,078.89	73,079.11	73,080.23	73,081.48	73,081.75	73,082.02	73,082.29	73,082.58	73,082.87
2.00	17,814.46	17,814.71	17,814.96	17,815.22	17,815.48	17,815.75	17,817.16	17,818.72	17,819.05	17,819.39	17,819.74	17,820.10	17,820.46
2.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Final	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Sensitivity to solar radiation

8:13 PM 2/11/2019 Table 3 (Sensitivity to solar radiation)

Months	1: Simulated	2: Simulated	3: Simulated	4: Simulated	5: Simulated	6: Simulated	7: Simulated	8: Simulated	9: Simulated	10: Simulated	11: Simulated	12: Simulated	13: Simulated
.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00
.25	372,884.28	372,884.14	372,904.01	372,923.88	372,943.74	372,963.61	373,082.94	373,162.27	373,182.13	373,202.00	373,221.87	373,241.73	373,261.60
.50	354,416.03	354,604.42	354,792.81	354,981.19	355,169.58	355,357.97	356,299.91	357,241.85	357,430.24	357,618.63	357,807.02	357,995.41	358,183.80
.75	319,756.78	320,262.35	320,767.92	321,273.49	321,779.05	322,284.62	324,812.46	327,340.29	327,845.86	328,351.43	328,856.99	329,362.56	329,868.13
1.00	269,010.72	269,982.12	270,953.52	271,924.92	272,896.33	273,867.73	278,724.74	283,581.75	284,553.15	285,524.56	286,495.96	287,467.36	288,438.76
1.25	202,269.54	203,855.43	205,441.33	207,027.22	208,613.12	210,199.01	218,128.48	228,057.95	227,643.84	229,229.73	230,815.63	232,401.52	233,987.42
1.50	119,580.17	121,929.21	124,278.25	126,627.29	128,976.33	131,325.38	143,070.58	154,815.79	157,164.83	159,513.88	161,862.92	164,211.96	166,561.00
1.75	42,457.81	45,520.05	48,582.30	51,644.54	54,706.78	57,769.02	73,080.23	88,391.45	91,453.69	94,515.93	97,578.17	100,640.41	103,702.66
2.00	0.00	0.00	0.00	0.00	0.00	0.00	17,817.16	35,948.13	39,574.32	43,200.52	46,826.71	50,452.91	54,079.10
2.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1,460.02	5,500.92	9,541.81	13,582.71	17,623.61
2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Final	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Sensitivity to Ammonia

8:45 PM 2/11/2019 Table 5: p2 (Sensitivity to Ammonia)

Months	1: Simulated	2: Simulated	3: Simulated	4: Simulated	5: Simulated	6: Simulated	7: Simulated	8: Simulated	9: Simulated	10: Simulated	11: Simulated	12: Simulated	13: Simulated
.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00	375,000.00
.25	373,062.86	373,062.87	373,062.88	373,062.88	373,062.89	373,062.90	373,062.94	373,062.98	373,062.99	373,062.99	373,063.00	373,063.01	373,063.02
.50	356,299.44	356,299.49	356,299.54	356,299.58	356,299.63	356,299.68	356,299.91	356,300.15	356,300.20	356,300.24	356,300.29	356,300.34	356,300.39
.75	324,811.20	324,811.33	324,811.45	324,811.58	324,811.70	324,811.83	324,812.46	324,813.08	324,813.21	324,813.34	324,813.46	324,813.59	324,813.72
1.00	278,722.19	278,722.45	278,722.70	278,722.95	278,723.21	278,723.46	278,724.74	278,726.02	278,726.28	278,726.53	278,726.79	278,727.05	278,727.30
1.25	218,124.06	218,124.50	218,124.94	218,125.38	218,125.83	218,126.27	218,128.48	218,130.70	218,131.14	218,131.58	218,132.03	218,132.47	218,132.92
1.50	143,083.69	143,084.38	143,085.07	143,085.76	143,086.44	143,087.13	143,070.58	143,074.04	143,074.73	143,075.43	143,076.12	143,076.81	143,077.51
1.75	73,070.91	73,071.84	73,072.77	73,073.71	73,074.64	73,075.57	73,080.23	73,084.91	73,085.84	73,086.78	73,087.72	73,088.65	73,089.59
2.00	17,805.91	17,807.03	17,808.16	17,809.28	17,810.41	17,811.53	17,817.16	17,822.80	17,823.93	17,825.06	17,826.19	17,827.32	17,828.45
2.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.25	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.50	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Final	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

APPENDIX II

Table 10 Additional data for calibration

Year	BOD inflow (mg/l)	PH inflow (mg/l)	NH ₃ inflow (mg/l)	T out flow (°C)	FC inflow (FC counts/100ml)
2002	16	7.3	59.3	18	1090
	166	7.7	10	23	10975
	40	7.9	22.5	17.7	1097
2005	316	7.4	47.5	22.3	2254
	58	7.61	8.54	18	100000
	144	7	1.79	18	80000
2009	247	7.4	4.04	17	100000
	142	7.3	0.12	24.3	190000
	244	7.64	0.11	22	900000
2011	57	6.8	10	23.9	11000000
	36	7.08	6	17	10000
	203	7.31	7.5	15.7	2254
2017	232.5	6.3	9.49	21	2254

Table 11. WHO quality guidelines for wastewater use in agriculture

Parameter	Unit	Irrigation of vegetables likely to be eaten Cooked	Irrigation of vegetables likely to be eaten un cooked	Irrigation of ornamental fruit trees and fodder crops
BOD5	mg/l	30	<20	<240
COD	mg/l	100	-	-
DO	mg/l	>2	-	-
TSS	mg/l	500	<20	<140
pH		6-9	-	-
Turbidity	NTU	10	-	-
NO ₃	mg/l	30	-	-
NH ₃	mg/l	-	5	-
T-P	mg/l	-	2	-
T-N	mg/l	-	15	-
FC	Faecal coliform per 100 ml	1000	<200	<1000

Table 12. Parameters used in the model

Symbols	Units	Values	Source	Descriptions
BOD _{5in}	mg/l	Collected data	Study	Inflow Biochemical Oxygen Demand at 20 °C in 5 days
K _{BOD}	mg/l	60	(Metcalf & Eddy., 1979)	Half-saturation constant of BOD
K _{DO}	mg/l	1.3	(Kayombo et al., 2005)	Half-saturation constant of DO
NH _{3in}	mg/l	Collected data	Study	In flow Ammonia-Nitrogen
K _{PH}		220	(Kayombo et al., 2005)	Half-saturation constant for pH
K _{NH}	mg/l	1	(Kayombo et al., 2005)	Half-saturation constant of ammonia
μ (or gr)	d ⁻¹	Calculated	(John et al, 2012)	Specific Growth rate
FC _g	FC counts/100 ml.d	Calculated	(Metcalf & Eddy., 1979)	Faecal Coliform in growing process
FC _i	FC counts/100 ml.d	Calculated	(Metcalf & Eddy., 1979)	Inflow Faecal coliform
Q _i	m ³ /d	Collected data	Study	Inflow rate
V _{cw}	m ³	Calculated	(Kadlec & Wallace, 2009)	Volume of VSSFCW
μ _{max}	mg/l	0.95	(John et al,2012)	Max specific growth rate
L	m	2.1	(Kadlec & Wallace, 2009)	Length of VSSFCW

w	m	0.7	(Kadlec & Wallace, 2009)	Width of VSSFCW
H	m	0.3	(Kadlec & Wallace, 2009)	Substrate depth of VSSFCW
FC _{IP}	FC counts/100 ml.d	Calculated	(Kimwaga et al., 2003)	Faecal coliform in the inflow process
a	m	0.15	(Kimwaga et al., 2003)	Constant
b	m	0.95	(Kimwaga et al., 2003)	Constant
f	m/d	1	(Kimwaga et al., 2003)	Filtration rate
FC _f	FC counts/100 ml.d	Calculated	(Kimwaga et al., 2003)	Faecal coliform in filtration process
s _r	m/d	0.0864	(Metcalf & Eddy., 1979)	sedimentation rate
FC _{sd}	FC counts/100 ml.d	Calculated	(Kimwaga et al, 2003)	Faecal coliform in the sedimentation process
Φ (or OL)	m ² /cal	0.035	(Metcalf & Eddy., 1979)	Light mortality coefficient
I _{avg}	cal/m ² .d	4558-6450	Study	Average solar radiation
r _i	d ⁻¹	Calculated	(Kayombo et al., 2005)	Removal rate coefficient due to solar radiation
T	°C	Collected data	study	Temperature of the wastewater
ΦT (or OT)	No.	1.04	(Kayombo et al., 2005)	Temperature coefficient
r ₂₀	d ⁻¹	0.05-1.14	(Kayombo et al., 2005)	Removal rate at 20 °C
r _t	d ⁻¹	Calculated	(Kayombo et al., 2005)	Removal rate coefficient due to temperature

n	No.	Calculated	(Kimwaga et al., 2003)	Single collector removal rate coefficient
α (or x)	No.	$2.4 * 10^{-3}$	(Li Cheng, et al., 2006)	Sticking efficiency
u	m/d	Calculated	(Metcalf & Eddy., 1979)	Velocity of flow
Θ (or O)	No.	0.4	(Senzia et al., 2002)	Porosity of the bed
r_f	d^{-1}	Calculated	(Kimwaga et al., 2003)	Removal rate coefficient due to combined effects of adsorption, filtration and sedimentation
r_z	d^{-1}	Calculated	(Kimwaga et al., 2003)	Overall removal rate coefficient
FC_d	FC counts/100 ml.d	Calculated	(Kimwaga et al., 2003)	Fecal coliform due to natural die-off process
Q_o	m^3/d	Collected data	study	Out flow rate
FC_o	FC counts/100 ml.d	Collected data	study	Out flow fecal coliform
FC_{op}	FC counts/100 ml.d	Calculated	Kimwaga et al., 2003)	Faecal coliform in the outflow process
DO_o	mg/l	Collected data	Study	Dissolved Oxygen at the out let
PH_o	mg/l	Collected data	Study	pH values at the out let

