THESIS

GRAYWATER REUSE GUIDANCE AND DEMONSTRATION USING A CONSTRUCTED WETLAND TREATMENT SYSTEM

Submitted by

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ABSTRACT

GRAYWATER REUSE GUIDANCE AND DEMOSTRATION USING A CONSTRUCTED WETLAND TREATMENT SYSTEM

Communities throughout the United States and abroad are developing interest in innovative approaches to sustaining their freshwater resources. One method, graywater reuse for non-potable demands, is gaining popularity because it allows the reuse of minimally contaminated wash water generated at the home/office for non-potable demands, which then reduces the demand for treated water and preserves source waters. Graywater is defined as any wastewater generated at the home or office excluding water from the toilets, kitchen sinks, and dishwasher, but includes wastewater from the laundry, shower, and bathroom sinks. When compared to other wastewater generated in the home graywater is minimally contaminated with lower concentrations of organics, solids, nutrients, and pathogens, thereby rendering the water suitable for reuse with minimal treatment when compared to other domestic wastewater sources. Despite widespread interest in this innovative approach information on the separation and design of residential and/or commercial scale graywater systems have been limited. The objective of this study was 1) to provide a graywater reuse manual for home or business owners interested in separating sources of graywater from blackwater for graywater reuse and 2)

to determine the first order removal rates (*k*) of graywater constituents using both a free water surface (FWS) and subsurface flow (SF) constructed wetlands, in order to provide design guidance for future constructed wetlands that will be used to treat graywater.

Information regarding the separation and reuse of graywater is important to the success of graywater reuse systems. This thesis provides information to business and home owners about the separation of graywater from blackwater for graywater reuse. Part one of this thesis outlines the methods and equipment needed to install a dual plumbing system for the purpose of graywater reuse. Part one also describes how to design an individual graywater reuse system specific to the needs of the home or business owners, the technologies and equipment necessary for graywater reuse systems, known maintenance requirements for graywater systems, and best management practices to ensure safe reuse of graywater.

Individual graywater reuse systems for the home or office are too small to treat large amounts of graywater produced by residential neighborhoods or communities. Consideration should be given to treatment options that can handle and treat a large amount of graywater. Constructed wetlands can offer a scalable, economically sound, low tech and easily maintained method of treating graywater for large scale irrigation reuse. While constructed wetlands are an appropriate technology for graywater treatment there is little research providing the removal rates for the design of constructed wetlands for graywater reuse. Determining removal rates is important for creating wetland design standards for graywater treatment and reuse. Part two of this thesis provides the experimental results for determining the seasonal flow adjusted removal rates (*k*) of graywater constituents using a free water surface (FWS) constructed wetland and a

subsurface flow (SF) constructed wetland. Removal rates were evaluated over a two year period (2008-2010) for a FWS wetland and evaluated over the summer/fall of 2010 for a SF wetland. The results for the FWS included the biochemical oxygen demand (BOD$_5$) removal rates of 15.9 (m yr^{-1}) for summer removal, 15.2(m yr^{-1}) for fall removal, and 5.6 (m yr^{-1}) for winter/spring removal. The total nitrogen (TN) removal rates were 16.4 (m yr^{-1}) for summer removal, 8.5(m yr^{-1}) for fall removal, and 5.5 (m yr^{-1}) for winter removal. The total organic carbon (TOC) removal rates were 10.4 (m yr^{-1}) for summer removal and inconclusive for the TOC removal in the fall and winter seasons. The results for the SF during the summer included a BOD$_5$ removal rate of 19.1 (m yr^{-1}), a TOC removal of 22.8 (m yr^{-1}), a TN removal rate of 21.3 (m yr^{-1}), and an ammonia removal rate of 32.6 (m yr^{-1}). The results were inconclusive for the fall season due to a limited amount of data. When compared to other literature *k* values for sizing wetland for agricultural and municipal wastewater, results from this study had lower *k* values for BOD, which resulted in a larger required surface area (SA) for wetland design. The TN and ammonia *k* values were comparable to other literature design values.

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TABLE OF CONTENTS

TABLE OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

1.1 Water Conservation

Water is our most valuable resource and must be protected. The quantity of freshwater available for our use is limited, and the global demand for freshwater will increase due to an increase in developing nations whose increasing population will have an expectation of clean water. Currently the quality of freshwater in streams and lakes are declining due to increase pollution from point sources, including wastewater treatment plants, and non-point source pollution including agricultural runoff. To sustain enough freshwater to continue to meet demand, the need for greater water conservation efforts including a more efficient use of water must be implemented worldwide. Water conservation efforts, including water reuse, are necessary to sustain population growth, agricultural needs, recreational, and industrial uses.

Methods used to protect our freshwater sources include conservation and water reuse efforts. Water conservation focuses on reducing the amount of water used by installing low flow water fixtures and installing landscape concepts that require little to no water. Water reuse focuses on reusing non-potable water, including graywater and reclaimed wastewater, for non-potable applications such as irrigation, car washing, laundry, and toilet flushing. Wastewater reuse is an important strategy for conserving water resources.

One reuse application that conserves our freshwater sources is the practice of separating graywater from other wastewater sources and reusing the graywater to meet non-potable demands, such as irrigation or toilet flushing. This reuse application conserves our freshwater because it allows non-potable water to meet non-potable demands. This application allows the home/business owner to benefit directly from water reuse and allows the participation of individuals interested in the reuse without incurring large infrastructure costs.

The practice of graywater reuse is already implemented in other countries including Australia, Asia, Europe, and parts of the Middle East. In most cases graywater reuse has been implemented out of necessity, while others implement graywater reuse as a conservation effort. Australia conservation and reuse guidelines were developed due to a severe drought that limited fresh water sources and forced the government to establish conservation and reuse policies (Pinto *et al.* 2010). Japan's widespread graywater reuse applications are a result of its high population in a concentrated area (Al-Jayyousi 2003).

In the United States graywater reuse is rapidly gaining popularity in many semi-arid states and may become more common as water shortages develop into a national issue. Currently Arizona, California, Florida, Idaho, Nevada, New Mexico, North Carolina, Texas, and Washington have graywater reuse regulations, with other states considering its application. Within the states that allow graywater reuse, the regulations can vary significantly. California, the first state to develop graywater reuse regulations in 1994, is now considering financial support for the public in order to meet the water conservation goals established in a Memorandum of Understanding among the California water agencies (Sheikh 2010). Other states (including Arizona and New Mexico) promote graywater reuse and provide subsidies for installation of the graywater reuse systems. North Carolina only allows reuse of graywater if it is treated to water reclamation standards while other states limit the application of graywater to disposal into leach fields.

Additional states have looked into developing regulations for graywater reuse but have not implemented graywater reuse standards due to the lack of information on graywater reuse and concerns about public health issues relevant to graywater reuse. Those in the regulatory community often feel overwhelmed as they attempt to develop pragmatic regulations for safe graywater reuse, which is a result of the lack of peer reviewed information on graywater reuse systems.

Water shortages, either due to population increase or environmental conditions, serve as a motivation for water reuse. However, water reuse should not be just a response to a problem but should be implemented based on the recognition of the value of water as a resource and the effort to protect it for future uses.

1.2 Water Use

In the United States potable water is commonly used for non-potable demands such as irrigation, toilet flushing, and other uses that do not require the use of drinking quality water. In a study conducted by the American Water Works Association Research Foundation (AWWARF) the average US resident uses approximately 169.3 gallons of fresh water per capita per day (Mayer 1999). Indoor water use accounted for approximately 69.3 gpcd (42% of total water use), including an average of 18.5 gpcd for toilet flushing. Outdoor use varies with climate and yard size, but on average 100.8 gpcd (58% of total water use) was used for non-potable applications including irrigation.

Using potable water for toilet flushing and irrigation can be expensive and is a waste of a valuable resource. With approximately 70% of our potable water being used for irrigation and toilet flushing a substantial amount of our freshwater can be conserved by using graywater to meet a portion of these non-potable demands. On average, graywater is generated at a rate of 31.4 gpcd (45% of total indoor water use), and can typically meet toilet flushing demands and/or a portion of irrigation demands. Potential potable water savings from graywater reuse is estimated to be 21% for landscape irrigation, 20% for toilet reuse, and 31% for a combined irrigation and toilet reuse system (ChristovaBoal *et al.* 1996).

1.3 Graywater

Graywater is wastewater from a home or office excluding water from the toilets, kitchen sinks, utility sinks, and dishwasher, but includes wastewater from the laundry, shower, and bathroom sinks. Kitchen sinks and dishwashers are excluded as a graywater source due to an increase in potential for pathogens and high organic content which then leads to oxygen depletion and an increase in microbial activity in graywater (Roesner *et al.* 2006). Utility sinks can be contaminated with oils, paints, grease, which can be toxic to plants and have the ability to clog graywater filters, and are therefore not included as a graywater source. When compared to domestic wastewater, graywater is minimally contaminated with lower concentrations of organics (indicated by COD and BOD_5

levels), solids, nutrients, and pathogens (Table 1.1), which renders the water suitable for reuse with little treatment compared to domestic wastewater.

Table 1.1. Composition of Graywater to Domestic Wastewater

	Graywater Range[1] (mg L^{-1})	**Domestic Wastewater Range[2] (mg L^{-1})**
Chemical Oxygen Demand (COD)	77-240	250-1000
Biochemical Oxygen Demand (BOD_5)	26-130	110-400
Total Suspended Solids (TSS)	7-207	100-350
Total Nitrogen (TN)	0.36-0.64	20-85
Total Phosphorus (TP)	0.28-0.779	4-15
Total Coliform (CFU/100mL)	6.0×10^3-3.2×10^5	10^6-10^9
E. Coli (CFU/100mL)	<100-2800	

[1] (Mayer 1999) [2] (Tchobanoglous *et al.* 2003)

Although graywater is not as contaminated as domestic wastewater it does contain indicator organism concentrations in excess of standard concentrations permitted for recycled drinking, bathing, and surface irrigation water (Sheikh 2010). For graywater reuse to be effective, graywater should be treated to levels that are safe for human contact when exposure is likely (i.e. toilet reuse).

1.4 Current State of Knowledge on Graywater Reuse

Wastewater reuse is emerging as an essential part of a plan for effective management of water because it promotes preservation of freshwater supplies, potentially reduces pollutants in the environment, and reduces total costs in treating water and wastewater (Jefferson *et al.* 2001). Within a domestic residence graywater reuse represents the largest potential source for reuse (Al-Jayyousi 2003). However, improper reuse methods could potentially result in the spread of illness and/or unintended effects on the environment. Therefore, when reusing graywater, standards and best management practices must be implemented to safely reuse graywater and to avoid contaminating water with unsafe constituents.

Graywater is not of potable water quality and must be treated as such. Graywater may contain harsh chemicals including oils, paints, solvents, and heavy metals such as zinc and copper (ChristovaBoal *et al.* 1996) or contain boron that can be harmful to plant health even at low concentrations (Madungwe and Sakuringwa 2007). In addition, a number of studies have identified unsafe levels of indicator organisms in graywater (ChristovaBoal *et al.* 1996; Novotny 1990; Rose *et al.* 1991). The physical and chemical properties of graywater are highly variable depending on the source and are influenced by many factors including the number of household occupants, types of cleaners and personal care products used, grooming and hygiene habits, and sink disposal practices (Eriksson *et al.* 2002). These studies support the hypothesis that graywater may contain constituents that are harmful to human health and the environment. However, these studies also reinforce the implementation of safe handling and best management practices in order to safely and effectively reuses graywater. The following sections will discuss the current state on knowledge for graywater practices, dual plumbing systems, and long term effects of graywater.

1.4.1 Current Graywater Practices

A study conducted by the Soap and Detergent Association concluded that 7% of US homes were reusing graywater (NPD Group 1999). The study classified most of those graywater reuse systems as having been done without following the permitting process or creating construction drawings (Sheikh 2010). Additional homes may be practicing graywater reuse due to the changes in graywater information and graywater regulations from the time the study was completed in 1999. It is estimated that approximately 2% of the systems in that study were legally installed, indicating that the majority of graywater systems were installed without permits or construction drawings and were perhaps against the laws of the jurisdiction. Concerns exist regarding these unlicensed and possibly illegal systems. In order to alleviate some of those concerns more information about safe graywater practices needs to be provided for sustainable and safe graywater reuse practices.

Toilet flushing reuse systems have become popular in Europe and in developing countries such as India. Environmental and economic factors influence the installation of a graywater reuse system for toilet flushing that was installed in hotel in Spain (Gual *et al.* 2008; March *et al.* 2004). India is currently evaluating graywater criteria including treatment for reusing shower water to flush toilets in schools (Godfrey *et al.* 2010) and potential saving for reusing graywater for toilet flushing and irrigation (Mandal *et al.*

2011). Graywater reuse for toilet flushing is still relatively new concept in the United States and there was no information found during the literature review.

1.4.2 Dual Plumbing Systems

The concept of graywater reuse is relatively new in the US and there is limited research in the area of graywater separation. However, information is available through the world wide web. Unfortunately much of this information is not peer reviewed and is sometimes misleading. Most information on the internet advocates “green-do-it-yourself” systems or highlights information supplied by equipment providers selling products for graywater capture, storage, and application. Currently there are limited amount of peer reviewed sources where those interested can find more information on graywater separation and reuse.

Plumbing codes have limited information on the installation of dual plumbing systems and method for graywater reuse. The Uniform Plumbing Code (UPC) limits graywater system installation to single family homes for the septic system disposal. The UPC also requires the graywater system be regulated and permitted under the authority with regional jurisdiction. The UPC standards are written for septic systems advocating the disposal of the generated graywater rather than for the purpose of irrigation reuse. The International Plumbing Code (IPC) allows for broader reuse to include subsurface landscape irrigation systems but also advocates disposal rather than irrigation reuse. In addition to its standards on subsurface landscape irrigation systems the IPC has standards for toilet reuse applications with approved disinfection, potable water backup, coloring aspects, and graywater identification criteria. However, these standards are limited in terms of guidance for design or installation of a graywater reuse system.

Art Ludwig (2006) created a detailed graywater reuse manual that focused on installation of a dual plumbing system for irrigation applications but was limited in the toilet reuse information. The manual leads potential graywater users through the process of building a graywater reuse system but does not sufficiently emphasize the dangers of using kitchen sources nor does the manual identify safety measures to limit exposure of graywater.

Graywater is an effective method of water reuse which homeowners can benefit from but information available for the safe reuse of graywater and the design of a dual plumbing system and installation of a graywater reuse system is limited. Additional research is required in these areas for reuse to be done safely and effectively.

1.4.3 Long Term Effects of Graywater for Irrigation

The City of Los Angeles, California conducted a study in 1992 about the soil characteristics of homes that used graywater irrigation. The results in that study revealed an increase in sodium levels and sodium adsorption ratio (SAR) in the area of graywater irrigation but the plant health appeared to be unaffected. What continues to be the concern are the long term effects of graywater reuse on soil, plants, and groundwater. Currently Water Environment Research Fund (WERF) and the Soap and Detergent Association (SDA) are funding research to determine the potential threats of graywater reuse to human health, and potential long term effects of graywater on plant health, soil chemistry, and microbiology. The first phase of this research, a literature review, has been published and concludes that there are knowledge gaps regarding the long term effects of graywater reuse and additional research is required (Roesner *et al.* 2006). The second phase of this research evaluates the long term (5+ years) effects of graywater irrigation on the changes in soil chemistry, soil microbiology, indicator organism, and impacts on residential landscape plants (Sharvelle *et al.* 2010).

1.5 Objectives

The objectives of this two part study are:

1. to provide a graywater reuse manual for home or business owners interested in separating sources of graywater from blackwater for graywater reuse and
2. to determine the first order contaminate removal rates *k* of graywater constituents that will be applied for the design of constructed wetlands for graywater treatment.

The concept of graywater reuse is still relatively new with only limited research in the area of graywater separation for reuse. This thesis provides an overview on installing a graywater reuse system with a detailed peer reviewed graywater separation manual (Bergdolt *et al.* 2011 Submitted). This manual has been submitted to the Water Environmental Research Foundation (WERF) and was written to inform regulatory professionals, homeowners, and business owners with information on source separation of graywater from blackwater for the purpose of graywater reuse. This manual provides guidance on safe separation, operation, and end uses of graywater. Chapter 2 provides a short summery of the manual serving as a fact sheet on the topic of graywater separation for reuse with a detailed copy of the guidelines provided through the Water Environmental Research Foundation (WERF) (Bergdolt *et al.* 2011 Submitted).

The second objective of this thesis is to determine the seasonal removal rates of graywater constituents. Determining removal rates is important for creating design standards for graywater treatment and reuse. Constructed wetlands are designed by determining the required surface area (SA) needed to treat the influent wastewater. The SA is designed based on the first order areal removal rate constant (k) and background concentration (C^*) as proposed by Kadalec and Knight (1996). The removal rates for a free water surface (FWS) wetland and a subsurface flow (SF) wetland were determined by evaluating the k under different hydraulic loading rates (HLR), and during different seasons.

The FWS wetland was evaluated over a two year period (2008-2010) and the SF wetland was evaluated over the summer and fall of 2010. This research provided a range of the parameter k for biochemical oxygen demand (BOD_5), total organic carbon (TOC), ammonia and total nitrogen (TN) as they apply to graywater.

CHAPTER 2

GRAYWATER SYSTEMS

This chapter provides a general overview of the methods and components necessary to construct a graywater reuse system and is a standalone document intended to serve as a fact sheet. A copy of the complete WERF document "Guidance Manual for Separation of Graywater from Blackwater for Graywater Reuse" is available through WERF. Information in this chapter is based on experiences with graywater reuse systems at the residential and apartment scale, field tests on graywater systems, and demonstration scale research projects including graywater reuse systems.

2.1 Graywater Reuse

Graywater reuse is the process of separating graywater from other waste sources and then storing, treating and using the graywater to supplement non-potable demands such as irrigation and toilet flushing. Graywater is collected in a home or business using a dual plumbing system. Dual-plumbing is an additional plumbing system that allows graywater to flow to the storage/treatment system while allowing blackwater to continue to flow to the sewer (Figure 2.1). Graywater can be reused for toilet flushing, irrigation (Figure 2.2), or a combination of both (Figure 2.3)

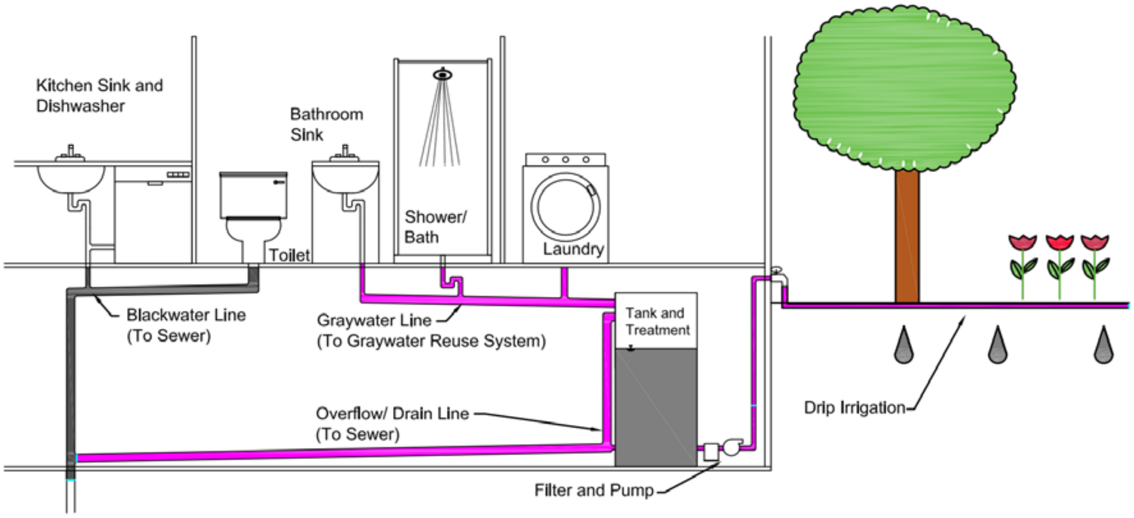

Figure 2.1. Graywater Reuse for Irrigation

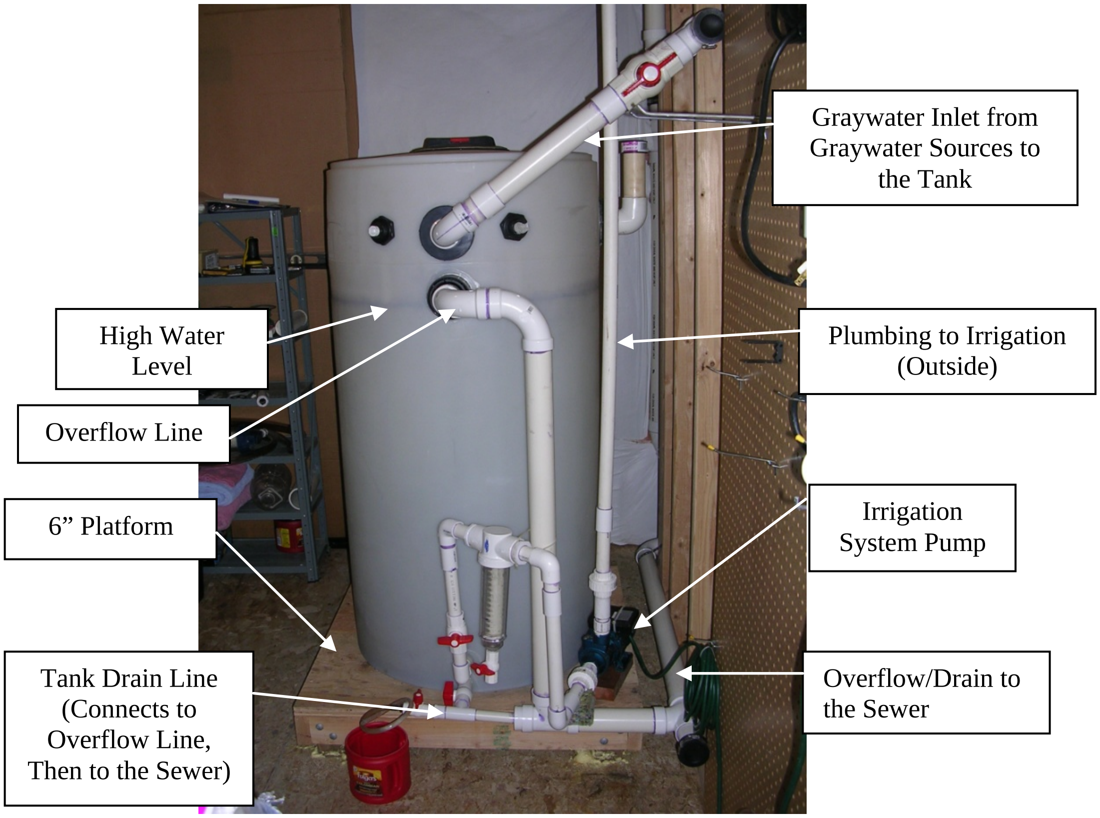

Figure 2.2 Residential Graywater Reuse System for Irrigation

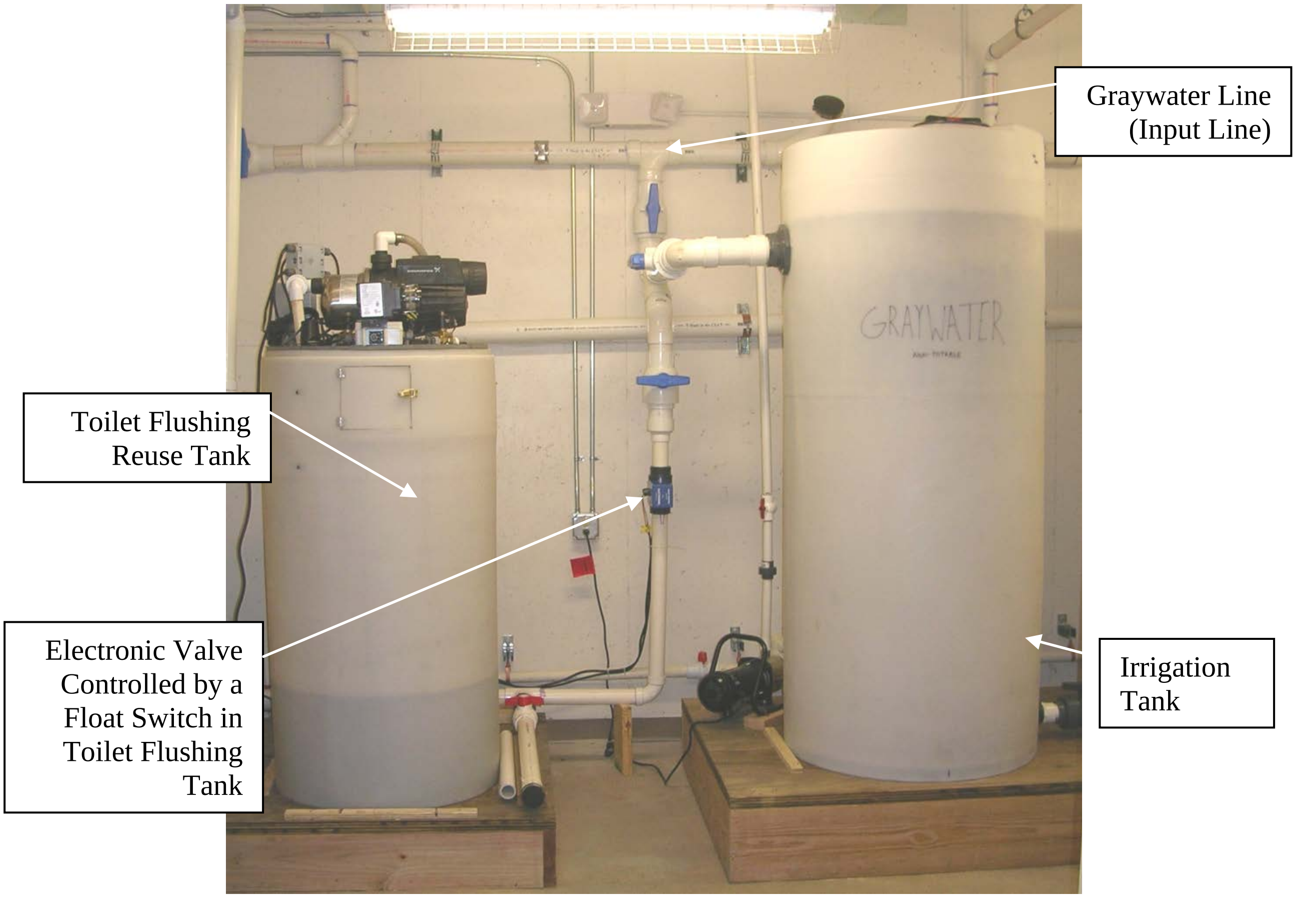

Figure 2.3. Hybrid System Including Graywater Reuse for both Toilet Flushing using a Water Legacy Toilet Reuse System and Irrigation Reuse System.

Reusing graywater can have several benefits including the overall reduction in potable water use and wastewater generation, and providing water and nutrients, without the use of synthetic fertilizer, to plants. In homes with septic systems, graywater separation and reuse can help decrease the capacity required for a septic system, which can result in smaller tanks, and less maintenance. Although graywater has many beneficial uses, graywater reuse is not appropriate for every home or situation. Installation of graywater reuse systems requires plumbing knowledge, investment in new equipment and materials, and possibly some additional construction costs to retrofit homes or offices. Additional considerations are the legality of graywater reuse since not all states allow graywater reuse.

The following steps should be followed for installing a graywater reuse system:

1. Determine the local laws and regulations for graywater reuse
2. Decide what to reuse the graywater for (irrigation, toilet flushing, or both)
3. Determine how much graywater you produce and whether it fits your reuse needs
4. Determine the plumbing installation, tank location, and required size of tank
5. Determine scheduled maintenance requirements

2.2 Consideration for Installing a Graywater Reuse System

Those interested in installing a graywater reuse system should be well informed on what is required to install and operate a graywater reuse system. Some considerations of importance include:

- *Local Graywater Laws.* Every state has different laws and regulations regarding the reuse of graywater. In some states graywater reuse is illegal. If the home or business owner is interested in installing a graywater reuse system, the owner should research local regulations on water reuse to understand the current laws and regulations on graywater reuse in their jurisdiction. If graywater is legal in that jurisdiction the state may require an approved permit application and inspection before system installation.

- *Maintenance.* Without proper maintenance and best management practices, graywater reuse can result in water quality issues that have potential health risks. Routine maintenance includes cleaning and replacing filters, replacing consumables (such as disinfection agents), turning off and emptying the system when the system is not in use for several days, winterizing the system, and other required maintenance based on the manufacture's recommendations. Proper maintenance ensures that the graywater system will work as intended over time, thus limiting health risks.
- *Existing plumbing*. Retrofitting plumbing for graywater reuse in an existing home can be cumbersome. Most homes and/or business owners require assistance from a licensed plumber. While separation of the plumbing into a dual plumbing system is simpler in single story homes with unfinished basements or crawl spaces, multi-story homes or homes with a finished basement may require some drywall removal and repair. It is important to evaluate the plumbing requirements for a specific home or office.
- *Graywater Generation.* When designing the system, the home or business owner will need to determine how much graywater his home/office generates and evaluate whether it is enough for the intended end use.
- *Desired end use.* Common end uses include drip irrigation and toilet flushing. All of the end uses require plumbing considerations, storage, and equipment that may include pumps, filters, and irrigation emitters. For toilet reuse systems, additional treatment including disinfection is required.

- *Treatment.* Since graywater contains organics, solids, nutrients, and pathogens it may require treatment before the graywater can safely be reused, depending on the end use of the water and the potential risk of exposure to humans and animals. For applications that limit exposure to humans, such as subsurface or drip irrigation, treatment may be as simple as a coarse filter to remove solids which may clog drip lines. If exposure is more likely, as is the case for reuse for toilet flushing, a filtration and disinfection system may be necessary.
- *Budget.* Installation of dual plumbing systems, graywater systems, tanks, pumps, and end use components can be expensive. Plumbing and manufactured systems can range from several hundred to several thousand dollars.

The decision tree in Figure 2.4 is a guide to understanding whether a graywater reuse system is appropriate for a home or business owner.

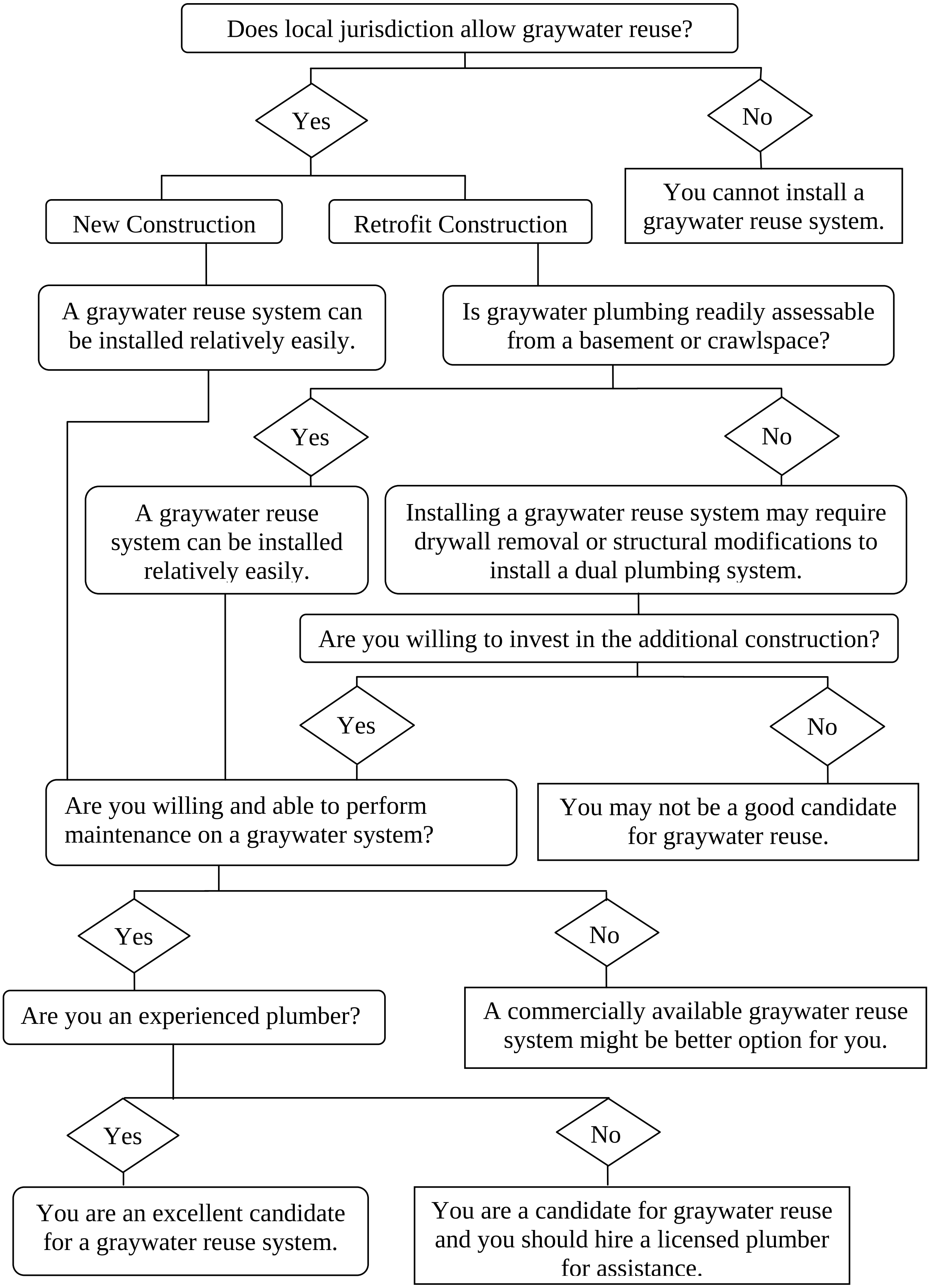

Figure 2.4 Decision Tree for a Graywater Reuse System in a Home or Business

2.3 Uses for Graywater

The two most common uses for graywater in the United States are to supplement irrigation demand and toilet flushing. Most home or business owners choose a single end use to for graywater rather than installing a system for more than one use. Creating a system for both irrigation and toilet flushing is possible, but due to the disinfection requirement for toilet flushing, separate tanks may be required for irrigation and toilet flushing reuse.

2.3.1 Reuse for Irrigation

Irrigation demand is typically the largest household water demand and on average is estimated to be about 100 gallons per capita per day or approximately 60% of your home's overall water use (Mayer 1999), depending on climate, region, irrigation area, and season (spring/summer months). The average person only generates enough graywater to meet a portion (30 gpcd) of their irrigation needs unless irrigated areas are xeriscaped. On average, xeriscaped and landscaped areas require less graywater than turf lawns.

Subsurface or drip irrigation is required for graywater reuse because spray irrigation (sprinklers) increases the opportunity for viruses and bacteria to become airborne, which increases the potential contact with humans or pets. Typically subsurface or drip irrigation reuse only require a coarse filtration system without disinfection. Irrigation of food crops is not recommended except for fruit trees, but fruit that has fallen to the ground should not be eaten.

2.3.2 Reuse for Toilet Flushing

On average, the amount of water used for toilet flushing is about 18.5 gpcd and the amount of graywater that is generated is approximately 31.4 gpcd (Mayer 1999), thus the graywater that is generated in a household typically exceeds the amount required for toilet flushing. However, toilet flushing demands may vary significantly depending on the amount of time spent in the home or office.

Reuse of graywater for toilet flushing typically requires installation of a more complex system compared to graywater reuse irrigation systems due to the requirement for disinfection, which is required since there is an increased potential for graywater to come into contact with humans and animals. Typical treatment consists of a combination of filtration and disinfection. The purpose of disinfection is to kill pathogens and bacteria that are present in the graywater. Disinfection options that are common for graywater reuse systems include ultraviolet (UV) light, chlorine, iodine, peroxide, and ozone. Commercially available systems are recommended for situations when graywater will be reused for toilet flushing, rather than do-it-yourself systems.

2.4 Components of a Graywater Reuse System

When designing a graywater reuse system you need to evaluate elevation differences within your plumbing, assess a tank location, and install a dual plumbing system to divert graywater to the reuse system tank or to the sewer.

2.4.1 Evaluating Elevation Differences within the Existing Plumbing

Elevation differences between the graywater fixtures, the storage tank, and the sewer main allows the graywater reuse system to work properly. The location of the graywater sources collected (i.e. showers, laundry, bathroom sinks) should be several feet higher than the top of the tank to avoid graywater from backing up into the fixtures. The location of the sewer main line should be considered when determining the location of the tank in order to install the required plumbing that allows the flow of excess graywater to the sewer. The overflow line must be located high enough above the sewer main to allow excess graywater to flow to the sewer. If the tank is on the same floor as the fixtures (i.e. basement or the lowest floor), the lack of elevation will not allow for gravity flow and a sump pump is required to collect the graywater.

2.4.2 Dual Plumbing System

Regardless of the end use (toilet flushing or irrigation) a graywater system requires dual plumbing. A dual plumbing system is plumbing which captures and diverts graywater for reuse while different plumbing allows blackwater to continue to flow to the sanitary sewer. When designing and installing a new graywater system it is important to understand how graywater plumbing can be separated. Fundamental to this is the understanding of basic indoor plumbing.

Potable water is provided to your home/office from the local utility company through a water meter or a well. Water is then distributed to the hot water heater as well as other fixtures inside the home or business. Water from the utility is pressurized which allows it to travel to different areas of the home or business. After potable water is used, wastewater is collected using the drain lines that flow to the main sewer line or septic system.

Installing a dual plumbing system requires access to existing plumbing, installing new collection lines to allow graywater to flow to the graywater reuse system and separating the new graywater lines from the existing blackwater lines which will continue to allow the blackwater to flow to the sewer main (Figure 2.5). This process requires detailed plumbing knowledge and specific tools.

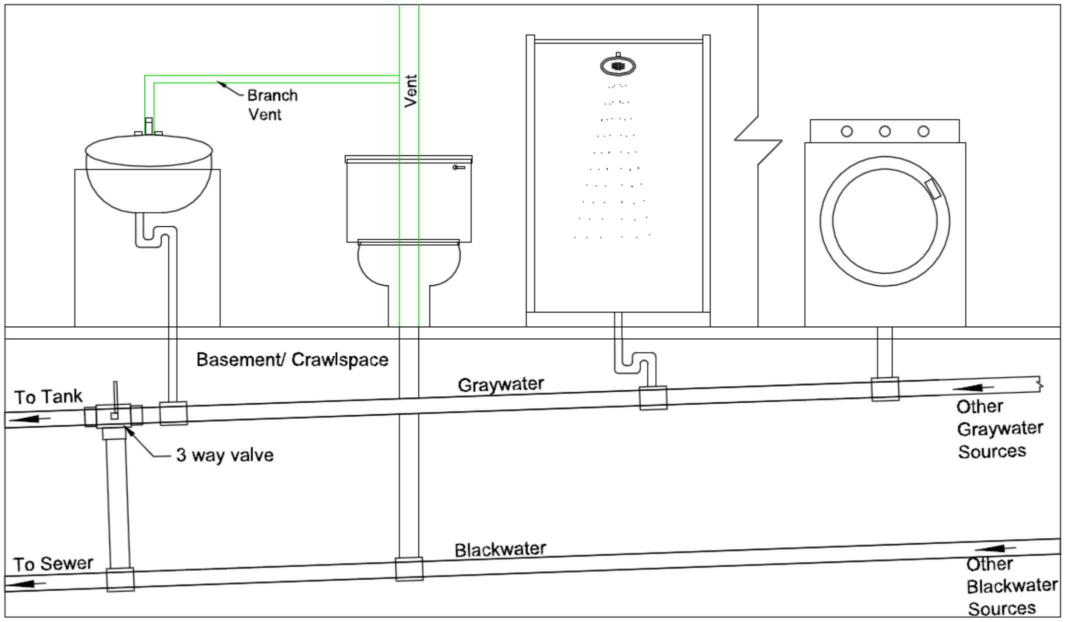

Figure 2.5 Graywater Separation Using a Dual Plumbing System

2.4.3 Storage and Treatment

Storage tanks allow you to collect and store graywater until it is needed for reuse. To ensure that the quality of the graywater does not degrade over time it is recommended that the graywater is used within 24 hours of being collected, depending on temperature, unless it receives additional treatment (i.e. disinfection or aeration).

Components of a graywater tank should include graywater inlet lines, overflow lines, drain lines, and a vent (Figure 2.6). *Graywater inlet lines* (Figure 2.6) are the plumbing that collects the graywater and conveys it to the tank. *Overflow lines* (Figure 2.6) are required in all graywater reuse systems to allow excess graywater in the graywater tank to flow back to the main sewer line. *Drain lines* (Figure 2.6) allow the graywater tank to be drained to the sewer for maintenance or when the water is not used for more than 3 days (e.g. when the family leaves for vacation). A valve should be placed on the drain line to turn off the flow to the sewer when the tank is in operation. Vents (Figure 2.6) are required in the tank to equalize pressure, prevent pressure buildup resulting from gas production, allow odors to dissipate outside, and allow the graywater system to perform properly. A backflow preventer may be required, depending on the local jurisdiction, between the tank and the sewer main to prevent any flow from the sewer main into the tank.

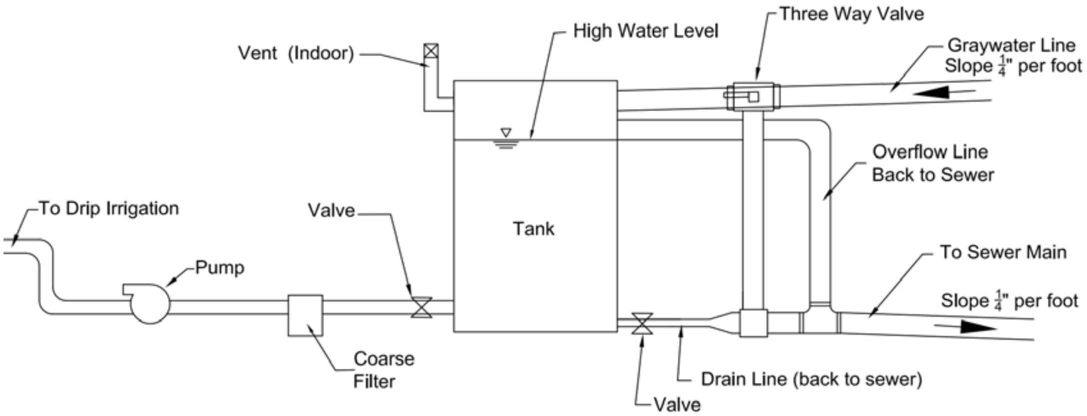

Figure 2.6 Graywater Reuse System for Irrigation

2.4.4 Graywater Best Management Practices

Graywater is not of potable quality as it contains constituents that pose health risks to people depending on the level of exposure. It is important to safely handle and reuse graywater to limit exposure to pathogens. When implemented correctly, best management practices are techniques that will reduce problems associated with a graywater reuse system and ensure the protection of human health and environmental quality. They are designed to increase safety, ease of use, and promote successful application of graywater reuse systems.

Best management practices for the handling of graywater for both irrigation and toilet flushing include:

- Ensuring that the graywater reuse system conforms to state and local graywater reuse guidelines.
- Using a licensed plumber early in the design (Masters Plumbers and Mechanical Services Association of Austrialia *et al.* 2008).
- Not using water from the kitchen/dishwasher/toilet with the graywater system (Arizona Department of Environmental Quality 2010).
- Labeling all pipes and outlets to indicate graywater plumbing (Arizona Department of Environmental Quality 2010; International Code Council 2009).
- Protecting potable water sources with backflow preventers and graywater identification labels (International Code Council 2009; Masters Plumbers and Mechanical Services Association of Austrialia *et al.* 2008)

- Not using the reused graywater to irrigate food crops except for fruit trees. This includes not eating fruit that has fallen to the ground (Arizona Department of Environmental Quality 2010; Masters Plumbers and Mechanical Services Association of Austrialia *et al.* 2008)
- Using soaps and detergents with low amounts of salt, boron, and phosphorus in them. Typically liquid detergents have fewer salts than the powder mixes. Where possible look for biodegradable detergents and soaps (Masters Plumbers and Mechanical Services Association of Austrialia *et al.* 2008).
- Covering and sealing the tank (Arizona Department of Environmental Quality 2010).
- Limiting human contact and exposure to graywater (Arizona Department of Environmental Quality 2010).
- Avoiding contact with mouth or face when performing maintenance on a graywater reuse system. Wash hands immediately after handing graywater (Masters Plumbers and Mechanical Services Association of Austrialia *et al.* 2008).

For a more detailed description of the separation of graywater from blackwater for graywater reuse, refer to the manual (Bergdolt *et al.* 2011 Submitted) provided through the Water Environmental Research Foundation (WERF).

CHAPTER 3

LITERATURE REVIEW ON CONSTRUCTED WETLANDS

This chapter provides a literature review on the treatment of wastewater using constructed wetlands including existing information on how to design constructed wetlands.

3.1 Introduction

Natural wetlands are one of the most biologically productive ecosystems on the planet and they can effectively treat most constituents found in municipal and agricultural wastewater (Kadlec and Wallace 2009). Constructed wetlands are designed to mimic the natural wetland treatment process through the construction of wetland vegetation, soils, and through the microbial processes used to treat the constituents found in wastewater (Babatunde *et al.* 2011). Constructed wetlands are artificial wastewater treatment systems consisting of shallow ponds, which have been built with impervious liners and planted with aquatic plants (EPA 2000). Constructed wetlands use a combination of biological, physical and chemical processes including sedimentation, precipitation, adsorption to soil particles, assimilation by plant tissue, and microbial transformations and interactions to treat the different constituents found in wastewater (Babatunde *et al.* 2011; EPA 2000).

The three most common types of constructed wetlands that treat wastewater are the free water surface (FWS), subsurface flow (SF) and vertical flow (VF) wetlands. FWS wetlands resemble natural wetlands because they consist of aquatic plants which root in a soil layer at the bottom of the wetlands. Water is treated as it flows through the roots and stems of the plants (EPA 2000). SF wetlands (also called vegetated submerged beds (VSB)) do not have standing water but rather wastewater flows through a bed of media (typically crushed rock, small stone, or sand), which are planted with aquatic plants. In a SF wetland, wastewater flows horizontally through the media and is treated as it comes in contact with the rhizomes of these plants. VF wetlands distribute wastewater across the surface of a sand or gravel bed and treatment occurs as the wastewater percolates through the plant root zone (Kadlec and Wallace 2009).

Constructed wetlands have been successful in treating a variety of wastewater sources including agricultural, mining, municipal, landfill leachate, urban storm water, and field runoff (Kadlec and Wallace 2009). The technology can offer several advantages over traditional treatment options including:

- Low capital cost (Davis 1995)
- Low operation and maintenance expenses (Davis 1995)
- Operation and maintenance on-site requirements only happen periodically rather than continuously (Davis 1995)
- Operates well when fluctuations in flow exist (Davis 1995)
- Scalability (Davis 1995)
- Serve as a habitat for many wetland plants and animals (Davis 1995)
- Can be built as part of the landscape (Davis 1995)

- Environmentally sensitive approach to water treatment and reuse (Davis 1995)

Natural wetlands have been used for treatment of wastewater disposal for as long as wastewater has been collected, with documented cases dating back to 1912 (Kadlec and Knight 1996). Research on using constructed wetlands as a treatment for wastewater began in Europe in the 1950’s. In the United States research for the treatment of domestic and municipal wastewater began in the late 1960’s and more recently has incorporated the treatment of agricultural and industrial wastewater, landfill leachate, and storm water runoff (Kouki *et al.* 2009).

In the 1980’s and 1990’s the Environmental Protection Agency (EPA) started cataloging different natural and constructed wetlands to create the North American Treatment Wetland Database (NADB). This database cataloged the use of natural and constructed wetlands in treating a variety of wastewater, which was the first step in understanding how these wetlands were able to treat a variety of wastewater streams. However, the EPA admits that the NADB contains questionable information and raw data that may not be applicable in determining the efficiency of wetland treatment (EPA website Accessed 2011), which means that the data is not adequate for designing or modeling constructed wetlands (EPA 2000). The EPA has since updated the database and created the treatment wetland database (TWDB). This database contains more information about various constructed wetlands including the system descriptions, locations, size, and the constituents that were monitored. Although the TWDB database provides more information than the NADB, its usefulness is limited because it does not provide sufficient information to design a constructed wetland.

3.2 Constructed Wetlands Design

Although constructed wetlands have been treating wastewater since the 1950's, there was limited criteria in how to design a constructed wetland (EPA 1993). The EPA (1993) advised to keep design simple, with gravity flow, and provide for a contingency plan, but did not provide sizing or treatment criteria. Finally in 1996, Kadlec and Knight developed sizing criteria for the design of a constructed wetland using a first order removal rate constant (*k)* with background concentration (*C*)*. The *k-C** model assumes ideal plug flow conditions, which evenly distributes flow through the wetlands. This modified first order equation develops criteria for the design of a constructed wetland by providing a relationship between mass loading rates and expected treatment efficiencies.

Although the *k –C** model was successful in designing wetlands, in 2000 the EPA developed their own criteria for sizing both FWS and SF wetlands that treat municipal wastewaters using average loading rates (EPA 2000). The EPA loading rates can be universally applied to all areas of the country and across different climates and temperatures. These loading rates contain several safety factors within the model to allow for safe effluent conditions in various climates despite the influent concentrations.

Despite the EPA's suggested criteria for designing a constructed wetland, the *k-C** model is still widely used for the treatment of different wastewaters. However, the *k–C** model assumes ideal plug flow conditions which critics state fails to characterize the complex flow within a constructed wetland. Tracer tests have shown that constructed wetlands do not follow ideal plug flow conditions, but instead the flow is intermediate between a plug flow and a completely mixed system (Babatunde *et al.* 2011). Researchers have proposed more sophisticated constructed wetland performance models which simulate non-ideal hydraulics. Kadlec (2003) recommends a tank in series (TIS) or plug flow with dispersion (PFD) modeling method which uses tracer test and gamma distribution methods to determine the amount of dispersion/ short circuiting that occurs within a constructed wetland. Despite the development of TIS and PFD modeling, these modeling approaches have not been adopted by practitioners due to the complexity and the amount of data required to properly use them (Babatunde *et al.* 2011). In addition, these models do not incorporate unsteady external hydraulic loading from precipitation and evapotranspiration events, which have significant effects on wetland treatment performance. Rousseau *et al.* (20004) reviewed current wetland design approaches and determined that the first order plug flow *k–C** modeling remains the best method for evaluating treatment of wastewater through a constructed wetland despite the assumptions of an ideal plug flow conditions. This method continues to be supported through different literature evaluations (Knight and Kadlec 1999; Son *et al.* 2010; Stein *et al.* 2006).

3.3 Review of Studies on Constructed Wetlands to Determine *k* and *C**

Recent studies have reinforced the efficient treatment of wastewater using constructed wetlands for treatment. These studies have examined a variety of wastewaters, in different climates, within different counties, and have used both laboratory and field testing. Most of these studies evaluate wetlands on the *k-C** model (reported in m yr^{-1}or m day^{-1}), mean annual removal rates (reported in g m^{-2} d^{-1}), or on percentage of mass removal of various constituents.

Tanner *et al.* (1994, 1998) evaluated the treatment performance of a SF constructed wetland treating dairy effluent in New Zealand. Tanner *et al.* (1994) evaluated the nutrient removal (TN and TP) of a planted and unplanted SF wetland and determined that when the retention times of the wetlands increased from 2-7 days, it resulted in increased removal rates of the constituents in the planted and unplanted SF wetlands. TN removal increased from 12% to 36% and from 37% to 75 % in the unplanted and planted SF wetlands respectively. Tanner *et al.* (1998) applied the *k*–*C** model to the SF wetland under different hydraulic loading rates and evaluated the wetland over several seasons. The study determined that there are seasonal patterns within the wetlands and that the *k* values were lower and the *C** values were higher than reported in Kadlec and Knight (1996).

In 1995, the Gulf of Mexico Program (GMP), the Alabama Soil and Water Conservation Committee and the National Council of Pulp and Paper Industry for Air and Stream Improvement (NCASI) conducted a literature review, data base, and research synthesis on constructed wetlands treating animal waste throughout Canada, Mexico, and the United States to create the Livestock Wastewater Treatment Database (LWDB). The LWDB gathered performance data and used that data (flow rates and pollution concentrations) to characterize the constructed wetland treatment of livestock effluent in North America and created new design criteria using the *k* –*C** model for constructed wetlands treating animal waste (Knight *et al.* 2000). The LWDB provides general guidelines for the surface area required when creating a wetland for the treatment of livestock effluent.

Stone *et al.* (2000) conducted a study to determine the *k* of FWS constructed wetlands that were treating swine wastewater. The study found that *k* values were higher than the rate constants found in Kadlec and Knight (1996), but slightly lower, but within the acceptable range, of the average reported in the LWDB (Knight *et al.* 2000) for both TN and Ammonia.

Jamieson *et al.* (2007) studied *k* values in a FWS wetland in Nova Scotia, Canada to develop design criteria for constructed wetlands treating agricultural wastewater in cold climates. The study found significantly lower *k* values than the values reported in the LWDB or Stone *et al.* (2002) for the same type of agricultural effluent, which can be attributed to either the colder temperatures, higher BOD_5 influent, or a low hydraulic loading rate (HLR).

The *k-C** model has been affective for determining the design criteria in designing constructed wetlands but previous studies prove that *k-C** estimates can have different values depending on the influent source and characteristics of the wastewater. The *k-C** values are different for different types of wastewater sources. It is important to study the *k-C** values for each type of wastewater to determine the specific treatment effects of a particular constructed wetland.

CHAPTER 4

DETERMINATION OF REMOVAL CONSTANTS FOR A CONSTRUCTED WETLAND TREATING GRAYWATER

4.1 Introduction

Increasing efforts to conserve water resources have prompted treatment and reuse of graywater for irrigation and toilet flushing to supplement domestic supply. Graywater is defined as wastewater generated at the home or office excluding water from the toilets, kitchen sinks, and dishwasher, but includes wastewater from the laundry, shower, and bathroom sinks. When compared to domestic wastewater, graywater is contaminated with lower concentrations of organics solids, nutrients, and pathogens (Eriksson *et al.* 2002; Tchobanoglous *et al.* 2003), which renders the water suitable for reuse with smaller amounts of treatment compared to domestic wastewater. While graywater is generally less contaminated than domestic wastewater, treatment is required to meet guidelines for unrestricted irrigation or toilet flushing. Constructed wetlands have emerged as a potentially viable technology for treatment of graywater for reuse on a community or multi resident scale (Jokerst *et al.* 2011 submitted).

Constructed wetlands have been used since the 1950’s for the treatment of domestic and municipal wastewater and more recently in the treatment of agricultural and industrial wastewater, landfill leachate, and storm water runoff (Kouki *et al.* 2009). When compared to conventional treatment systems, constructed wetlands offer considerable potential for treatment of graywater because they provide a low cost and easy to operate and maintain method of treatment (Tanner 1994). Constructed wetlands are scalable and can be designed with the ability to treat a large amount of wastewater. They also effectively treat wastewater with varying influent flows that may occur on a multi residential scale (EPA 1993). Previous studies have proven constructed wetlands to be an effective method for removing a variety of contaminants in other wastewater streams (Dallas and Ho 2005; Frazer-Williams *et al.* 2008; Gross *et al.* 2007; Masi *et al.* 2010) including graywater constituents (Abdel-Shafy *et al.* 2009; Gross *et al.* 2007; Jokerst *et al.* 2011 submitted; Kadewa *et al.* 2010; Masi *et al.* 2010; Paulo *et al.* 2009; Sklarz *et al.* 2009)**}**.

Constructed wetlands are designed based on the required surface area (SA) needed to treat the wastewater influent constituents. The SA is calculated based on the first order plug flow *k–C** equation developed by Kadlec and Knight (1996) or using average loading rates (EPA 2000). The *k–C** model is considered a good method for evaluating the treatment of the wastewater through a constructed wetland (Babatunde *et al.* 2011; Rousseau *et al.* 2004). Within the *k–C** method the parameter *k* is a removal rate constant and provides a relationship between mass loading rates and expected treatment efficiencies. The parameter *C** provides a non-zero background concentration. The *k-C** method estimates can vary depending on the source of the influent and the characteristics of the wastewater. The *k-C** model assumes ideal plug flow conditions and evenly distributed flow through the wetlands.

Research has developed criteria for designing and sizing constructed wetlands for treating municipal and agricultural wastewater effluent using both the *k*-*C** method and average loading rates. Kadlec and Knight (1996) report a range of *k*-*C** values found from a wetland treating municipal wastewater including reporting *k* ranges for BOD, TN, and TP. The EPA developed criteria for sizing both free water surface (FWS) and subsurface flow (SF) wetlands that treats municipal wastewaters using average loading rates (EPA 2000).

Tanner et al. (1998), Knight et al. (2000) and Jamieson et al. (2007), showed that the design criteria used for treatment of municipal wastewater is not applicable to treatment of high strength agricultural wastewater. Tanner *et al.* (1998), using dairy effluent, determined *k*-*C** estimates for COD, TN, TP, and ammonia after being treated by a SF wetland. The Livestock Wastewater Treatment Database (LWDB) characterize the constructed wetland treatment of livestock effluent in North America and created new design criteria using the *k*–*C** model for constructed wetlands treating animal waste (Knight *et al.* 2000). Jamieson *et al.* (2007) studied *k* values in a FWS treating agricultural wastewater in cold climates. The study found lower *k* than previously reported for the same type of agricultural effluent, which can be attributed to either the colder temperatures, higher BOD_5 influent, or a low hydraulic loading rate (HLR).

While extensive guidance is available for the design of constructed wetlands for agricultural and municipal wastewater treatment (EPA 2000; Jamieson *et al.* 2007; Knight 1993; Knight *et al.* 2000), little guidance is available for the design of constructed wetlands for graywater treatment. When compared to graywater, agricultural and municipal wastewater effluent contains a significantly higher loading of organic constituents and nutrients. These constituents are readily available and easily degradable to microorganisms and plants found within a constructed wetland (Sharvelle *et al.* 2007). Graywater is not only more dilute in organic content compared to domestic wastewater, but the organic material is primarily surfactants which degrade more slowly than typical wastewater constituents (Sharvelle *et al.* 2007). It would be incorrect to use the same sizing criteria for graywater than what is used to size wetlands for agricultural and municipal wastewater sources. However, guidance is currently unavailable for the design of constructed wetlands specifically for graywater treatment.

The objective of this study was to determine the first order contaminates removal rates (*k*) of graywater constituents using both a FWS and SF constructed wetlands in order to determine suitable design criteria for constructed wetland treatment of graywater. To achieve this objective, the hydraulic loading rate (HLR) of an outdoor constructed wetland system for graywater treatment was varied over different seasons. The parameters *k* and *C** were determined for five day biochemical oxygen demand (BOD_5), total organic carbon (TOC), total nitrogen (TN), and ammonia over varying seasons. The intention was to provide guidance on sizing of constructed wetlands specific for graywater treatment.

4.2 Materials and Methods

4.2.1 Wetland Configuration.

Details of the constructed wetland design, initial operations, and flow monitoring is described in Jokerst *et al.* (2011). Briefly, the experiments were conducted in Fort Collins, Colorado on the Foothills Campus of Colorado State University (CSU). A pilot scale constructed wetland system that consists of a FWS and a SF wetland was used for this research (Figure 4.1). The wetlands are located outdoors in a semi-arid climate with no protection from temperature, rainfall, or evapotranspiration. The wetlands were constructed with distribution and collection headers to prevent short circuiting and to evenly distribute the flow (Figure 4.1- Figure 4.2). Both wetlands were constructed with impermeable ethylene propylene diene monomer rubber liners to prevent seepage or groundwater flux.

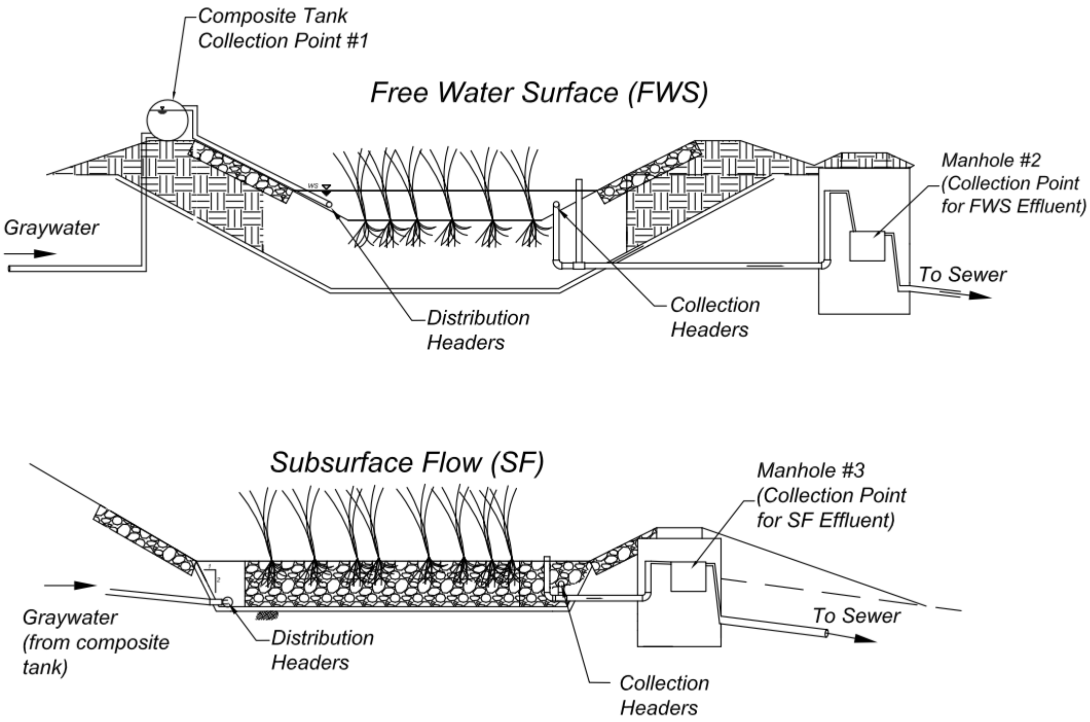

Figure 4.1 Wetland Configuration Skematic

The FWS wetland (Figure 4.2) was planted with cattails (*Typha latifolia*) and measured 9’ by 13’ (2.7m x 4.0m) with a depth of 14” (0.36m). The FWS wetland included a 1:2 rip rapped side slope. The volume of the FWS was approximately 530 gallons ($2.0m^3$), assuming an overall porosity of 0.8. A berm was constructed around both wetlands to minimize surface runoff into the wetlands.

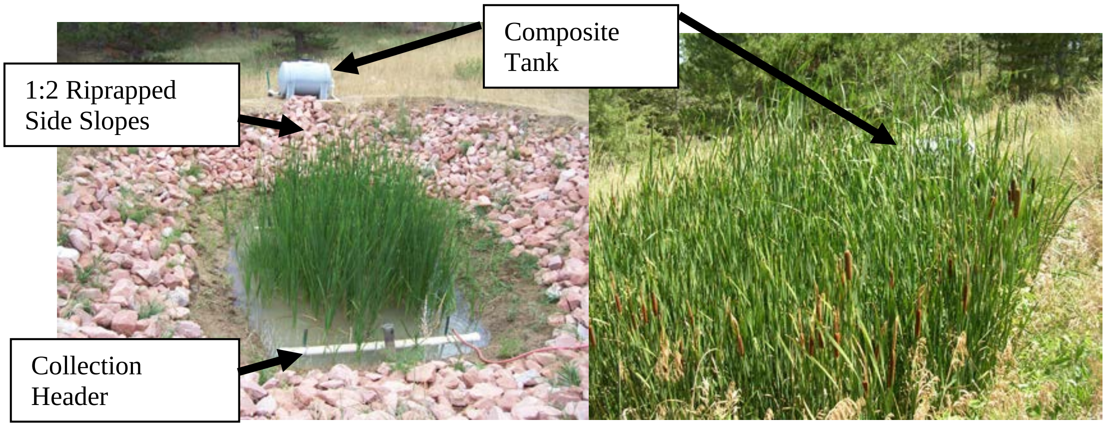

Figure 4.2 FWS in 2008 (left) and FWS in 2010 (right)

The SF wetland (Figure 4.3) was planted with hardstem bulrush (*Scipus acutus*) and measured 11’ by 17’ (3.3m x 5.2m) with a depth of 19” (0.5m). The SF wetland included a 1:1 rip rapped side slope. The SF wetland was filled with clean rounded native gravel with an average diameter of ½” (15mm). The SF wetland maintained a volume of 590 gallons (2.2m^3) considering an average porosity of 0.3.

Figure 4.3 SF in 2008 (left) and SF in 2010 (right)

The wetland system was constructed during the summer of 2007 and planting was conducted during the summer of 2008. Plants reached full maturity by the summer of 2009 (Figure 4.2 – Figure 4.3). From the summer of 2008 to the summer of 2010, the wetlands were operated in series. Raw graywater entered the FWS wetland and the effluent from the FWS flowed into the SF wetland (Figure 4.1). The wetlands were separated to operate in parallel during the summer and fall of 2010 to evaluate the performance of each wetland system separately. Removal rates for the SF wetland prior to the summer of 2010 are not reported in this study since the influent had undergone treatment from the FWS wetland before flowing to the SF wetland (Jokerst *et al.* 2011 submitted). The removal rates of the SF wetland are reported after the wetlands were separated and untreated graywater was supplied.

Water samples were collected at three locations to determine the quality of effluent and treatment provided by each wetland. The first sampling location was at the influent, before the graywater entered either the FWS or SF wetland (Figure 4.1). The second sampling location was the effluent from the FWS wetland. The third sampling location was the effluent from the SF wetland (Figure 4.1).

4.2.2 Graywater Sources

Graywater used for this experiment originated from three sources: directly from four lavatory sinks in the Atmospheric Chemistry Building (ACB) and from two different residential dormitories located on CSU’s main campus. The ACB was located south of the wetland and plumbed graywater, using a dual plumbed system, directly to the wetlands. Graywater production from the ACB fluctuated between 5 to 40 gallons per week depending on occupancy. Due to the low graywater flow rates additional sources were needed to conduct the experiment. Graywater from the first residence hall was collected using a retrofitted dual plumbing system installed during the summer of 2008 (Figure 4.4). The dual plumbing system collected graywater from the sinks and showers of 34 residences and conveyed the graywater to a 300 gallon storage tank located in the basement of the residence hall until the graywater was transported to the wetlands. During the summer of 2010 graywater was collected from an additional residence hall. The dual plumbing system for that additional residence hall collected graywater from fourteen showers and sinks used by twenty seven students and conveyed the graywater to two 300 gallon storage tanks located in the basement (Figure 4.4).

Figure 4.4 Graywater Collection Units Located in the Basement of the Residence Halls. (Right) shows 300 gallon tank in Edwards Dorm. (Left) shows a 300 gallon collection tank located in Aspen Hall.

As it was available, graywater was obtained from the residence halls for the experiment. Graywater was transported from the residence halls to the wetlands using a 500 gallon trailer Figure 4.5 and pumped to the wetlands using a variable speed peristaltic pump (Masterflex, Vernon Hills, Illinois). Graywater from all the sources entered into a singlestream within the first manhole and were further mixed within a composite tank (Figure 4.1- Figure4.2) before flowing into the wetlands.

Figure 4.5. 500 Gallon Trailor used to Transport Graywater from the Residence Halls to the Wetlands.

4.2.3 Flow Monitoring

Details of the flow monitoring system are described in Jokerst *et al.* (2011). Briefly, graywater flowed into each manhole (Figure 4.1) and was collected in a 1 gallon bucket. When the bucket was full, a float switch activated a submersible pump (Rule 25D, Gloucester, Massachusetts) which pumped the graywater through a turbine meter (Great Plain Industries TM100-N, Wichita, Kansas). Cumulative flow readings were monitored and recorded from the turbine meter 2-4 times per week depending on the set flow rate from the peristaltic pump located on the trailer. The volume of flow was divided by the lengths of time between readings to determine a flow rate.

4.2.4 Model Description

This experimental method involved monitoring water quality constituents including BOD_5, TN, ammonia, and TOC under varying HLR in order to estimate the first order k observed within the wetlands. The parameter k was estimated assuming a plug flow model and a non-zero background constant (C^*) as proposed by Kadlec and Knight (1996), utilizing Equation 4.1.

$$C_{eff\_predicted} = C^* + [e^{(-k/\mathrm{HLR})} (C_{in} - C^*)] \qquad \text{(Equation 4.1)}$$

Wherein:
$C_{eff\_predicted}$ = the effluent constituent concentration (mg l^{-1})
C_{in}= the influent constituent concentrations (mg l^{-1})
C^* = the background constituent concentration (mg l^{-1})

This model assumes a depth between 0.3m to 0.6m as recommeded by Kadlec and Knight (1996). The pilot wetlands have a depth of 0.36m (FWS) and 0.50m (SF).

However, Equation 4.1 does not account for dilution that may occur in a wetland due to precipitation events and concentration of constituents due to evapotranspiration (ET). Therefore, flow adjusted effluent concentrations were applied to account for water losses and gains through ET and/or precipitation. The approach applied by Tanner *et al.* (1998) to determine flow adjusted concentration was applied (Equation 4.2).

$$C_{eff,\,adj} = (Q_{out} / Q_{in}) * C_{eff} \qquad \text{(Equation 4.2)}$$

Wherein:
Q_{in} = flow into the wetland (m^3 day^{-1})
Q_{out} = flow out of the wetland (m^3 day^{-1})
C_{eff} = measured effluent constituent concentration (mg L^{-1})
$C_{eff,\,adj}$ = flow adjusted effluent constituent concentration (mg L^{-1})

If Equation 4.2 was not applied, precipitation events that occurred in the wetland would dilute effluent concentration. In addition, ET would decrease the Q_{out} and the measured concentration would be higher than what it should be if Equation 4.2 was not applied. Equation 4.2 accounts for both of the scenarios described above by adjusting the measured constituents accordingly.

4.2.5 Estimation of Area from *k*-*C** Parameters

Once the estimation of k and C^* are complete, they can be applied to design criteria for sizing constructed wetlands. The Surface Area (SA) of a wetland can be determined using Equation 4.3. Despite the literature reporting a high variability among k and C^* values, application of these parameters for wetland sizing has been found to be effective (Kadlec and Knight 1996; Kumar and Zhao 2011; Tanner et al. 1998).

$$SA = (Q/k) * \text{Ln}\,[(C_{in} - C^*)/(C_{eff\_adj} - C^*)] \qquad \text{(Equation 4.3)}$$

Wherein:
k = the first order removal rate constant (m day^{-1})
Q = flow rate (m^3 day^{-1})
C_{eff_adj} = the effluent constituent concentration (mg L^{-1})
C^* = the background constituent concentration (mg L^{-1})

The EPA has established separate guidelines for designing FWS and SF constructed wetlands for treatment of municipal wastewater. The EPA guideline designed a FWS system by using a system of fully vegetated zones followed by an open water zone. EPA suggests that the designs be based on the total area of the wetland. Wetland design is still based on required SA (Equation 4.4), only it is based on an Area Loading Rate (ALR), which, according to the EPA manual, is based on the constituent and the desired level of treatment.

$$SA = (Q * C_{in}) / \text{ALR} \qquad \text{(Equation 4.4)}$$

Wherein: ALR = Area loading rate (kg ha^{-1} day^{-1})

4.2.6. Experiment Design

4.2.6.1. Hydraulic Loading Rates

This experiment involved monitoring water quality constituents including BOD_5, TN, ammonia, and TOC under varying HLR in order to estimate the first order k observed within the wetlands. HLR is defined as the rate at which wastewater enters the wetland. HLR is the volumetric Q divided by SA (Equation 4.5) (EPA 2000).

$$\mathrm{HLR} = Q/\mathrm{SA} \qquad \text{(Equation 4.5)}$$

Constructed wetlands can be evaluated using the hydraulic retention time (HRT), which measures the amount of time that the constituents spend in the wetland, typically in days. Literature values for HRT can vary depending on the type of influent and desired treatment, but most of the results show a treatment range of 5-25 days, depending on influent contaminant concentration(Jokerst et al. 2011). HRT is Q divided by the total volume of the wetland (Equation 4.6).

$$\mathrm{HRT} = Q/V \qquad \text{(Equation 4.6)}$$

Wherein: V = the volume of the wetland (m^3)

For this experiment, to determine k, the wetlands were sampled under several different HLRs during each season. The k of constituents within the water directly correlate with HLR (Equation 4.1) The experiment was set up to obtain a baseline HLR of 0.036 ($m\ d^{-1}$, 5 day HRT) from the fall of 2008 to the summer of 2009. Once the baseline was established various samplings were conducted at varying HLRs (Table 4.1

and Table 4.2). HLRs were intentionally varied within seasons to obtain required data for the parameter estimation.

Table 4.1. HLRs for the FWS Wetland

Dates	Desired HRT (days)	Desired HLR (m day^{-1})	Observed HLR± SD (m day^{-1})
10/1/08 - 5/18/09	5	0.036	0.032 ± 0.005
5/19/09-8/20/09	7	0.026	0.027 ± 0.007
8/21/09-3/9/10	3	0.060	0.082 ± 0.039
3/10/10-5/13/10	7	0.026	0.027 ± 0.001
5/13/10- 8/22/10	-	-	-
8/23/10- 9/3/10	3	0.060	0.058 ± 0.005
9/4/10- 9/28/10	7	0.026	0.030 ± 0.012
9/29/10-10/13/10	4	0.045	0.046 ± 0.002
10/14/10-11/4/10	9	0.020	0022 ± 0.005

Table 4.2 HLRs for the SF Wetland

Dates	Desired HRT (days)	Desired HLR (m day^{-1})	Observed HLR± SD (m day^{-1})
7/30/1/10- 8/5/10	6	0.022	0.020 ± 0.003
8/6/10-9/3/10	4	0.037	0.037 ± 0.013
9/4/10- 9/28/10	3	0.043	0.047 ± 0.010
9/29/10- 11/4	7	0.018	0.019 ± 0.001

A range of HLRs were selected based on the amount of graywater that was available during each season and choosen to adequately estimate *k*-*C** parameters. This range was revised during the experiment depending on the results that were obtained from the baseline tests and the results obtained after each set of completed tests with a given HLR. A minimum of 3 sample points were desired at each HLR range. The desired HLR was set by varying the pump speed of the peristaltic pump described in section 4.2.2. Actual HLR was subjected to fluxuation due to climate conditions and the variability in the generation of graywater from both the dorm and building sources.

4.2.6.2 Water Quality Analysis Methods

Standard methods (APHA 1998) were used for all water quality analyses. Temperature and DO were analyzed in the field using a membrane electrode (Yellow Springs Instruments DO200, Yellow Springs, Ohio). Ammonia and pH were analyzed using an ion selective electrode (Thermo Scientific Orion 250A, Waltham, Massachusetts). Turbidity was measured with a nephelometric turbidimeter (Hach 2100N, Loveland, Colorado). TOC and TN were analyzed via combustion of acidified samples (Shimadzu TMN1, Columbia, Maryland).

All other measurements, sample collections, preparations, and storage methods were conducted following standard analytical methods (APHA, 1998). Quality assurance samples (blanks, duplicate analyses, and standards) were analyzed throughout the experiment. Multiple replications (generally 3) were used for every analysis when possible, and highest dilutions were always reported.

4.2.6.3 Definition of Seasons

For this experiment it was important to separate data based on seasons since mass removal rates of graywater constituents were determined to be significantly different for each season (Jokerst *et al.* 2011 submitted). This study was conducted in Fort Collins, CO, which is located next to the Rocky Mountains in a semi-arid region where large weather fluxuations can occur throughout the year. Seasons were determined using both average daily temperatures recorded at the National Climatic Data Center (NCDC) and effluent water temperatures measured at the FWS wetland (Table 4.3). Since this study used temperature as a criterion in separating the seasons and the seasonal temperatures can vary substantially in the Fort Collins, CO area, the seasons were not defined by a calendar year but were instead based on a range of temperatures and observations regarding the seasonal effects on the wetland's plants.

Table 4.3. Seasonal Dates, Effluent Water Temperatures, and Average Daily Air Temperatures

	Season Dates	**Mean Effluent Water Temp (°C)**	**Average Daily Air Temp (°C)**
Fall 2008	10/23/08 -11/19/08	8.6 ± 0.8	9.0 ± 3.5
Winter 2008-2009	11/20/08-4/13/09	5.3 ± 2.2	2.6± 6.0
Summer 2009	6/13/09-9/20/09	16.0 ± 2.4	20.3 ± 2.4
Fall 2009	9/21/09-10/27/09	7.7 ± 0.14	9.5 ± 6.2
Winter 2009-2010	10/28/09-4/11/2010	2.0 ± 0.94	1.50 ± 5.9
Summer 2010	5/17/2010-10/8/2010	16.0 ±3.8	20.4 ± 3.4
Fall 2010	10/9/2010-11/4/2010	9.6 ± 2.25	10.8 ± 2.5

The spring data was not sufficient to evaluate a *k-C** model. The data was limited in the spring season due to a short season and due to a limited amount of available graywater. The season was short because snow and freezing temperatures occurred as late as the middle of May (2010). The graywater was limited due to the students moving out of the residence halls during the spring season.

The summer season was defined when the average daily temperatures were consistently above 15°C and the wetland vegetation had established growth. The fall season was defined when the average daily temperatures ranged between 15-7°C and the leaves of the FWS turn yellow and brown in color. The winter season was defined when the average daily temperature was lower than 7.5°C. Plants in both wetlands typically started to grow in mid-May (spring), reached full maturity in July/ August and began to turn yellow in October. Ice thickness was measured for both the FWS and SF wetlands during the winter and resulted in a decreased volume in both wetlands. The formation of an ice layer reduces the depth of the water column, decreasing the volume, which then decreases the detention times, and will reduce HLR due to the reduction of SA.

4.2.7 Parameter Estimation

Estimates of *k* and *C** were evaluated over a range of HLRs by minimizing the least sum of squares error (SSerr) (Equation 4.7). Microsoft Excel's™ solver function solved for *k* and *C** by reducing the equation for SSerr (Equation 4.7) between the measured effluent constituent values and predicted effluent values described in Equation 4.1.

$$SSerr = \sum_{i=1}^{N} (C_{eff\_adj} - C_{eff\_predicted})^2 \qquad \text{(Equation 4.7)}$$

The pilot wetlands are a natural system and exposed to the atmospheric conditions and decaying plant matter. It was determined that a minimum background concentration needed to be set such that $C^* \neq 0$. Minimum and maximum background constraints were set for *C**. Upper and lower bounds for *C** were established based on literature values found or assumptions based on the experiment. Assumptions included setting the lower limits to less than actual C_{eff_adj} values. Upper and lower bounds for the *C** parameters for BOD_5 were 5-10 mg L^{-1}(EPA 2000; Kadlec and Knight 1996), and for TN and ammonia ranges were between 1-3 (mg L^{-1}) (Kadlec and Knight 1996). Parameters *k* and *C** were modified from their original estimates until the residual sum of square error (*RSSE*) between the measured and predicted effluent concentrations were minimized. The *RSSE* was divided by the number of measurements (*n*) in each data set to estimate a goodness of fit for the data.

Statistical Analysis: The C_{in} value affects the model fit curve for estimating k and C^* values (Equation 4.1). For the parameter estimation procedure, it is necessary to select one value for C_{in}, which would typically be an average value. Analysis showed that the ammonia and BOD_5 C_{in} values were determined to be significantly different between the summer of 2009 and the summer of 2010. C_{in} was compared between 2009 and 2010 by analysis of variance (ANOVA) with $p<0.05$ for significance using the data analysis pack in Microsoft Excel™. All other seasonal analyses were determined to be statically relevant ($p>0.05$) and analyzed together. It was unknown why there was a significant difference between the summer of 2009 and 2010.

4.2.8 Mass Removal of a Constructed Wetland

To evaluate performance of the wetland systems the mass removal of the constituents were calculated for both the FWS and SF wetlands. The average mass loading was calculated in order to account for differences in flow between the influent and the effluent due to precipitation and evapotranspiration events (Equation 4.8). Average mass loading rates equal to the product of the flow rate and the concentration of intrest, were averaged using the measured concentration and the flow rates immediately preceding the sampling event (Equation 4.8). Mass removal was computed by taking an average difference in the mass loadings between the influent and effluent concentrations, and dividing by the mass influent to determine the percentage removed (Kadlec and Knight 1996) (Equation 4.9).

$$\overline{\dot{m}} = \frac{\sum_{i=1}^{n}\left(C_i * \overline{q_i}\right)}{n}$$ Equation 4.8

$$MR(\%) = \frac{\overline{\dot{m}}_{Influent} - \overline{\dot{m}}_{Effluent}}{\overline{\dot{m}}_{Influent}} *100$$ Equation 4.9

Wherein: MR = the percent mass removal of the system
$\overline{\dot{m}}$ = the average mass loading rate (mg day^{-1})
n = the number of sampling events in a given season
C_{in} = the concentration of an influent or effluent for sampling event i (mg L^{-1})
$\overline{q}_i$ = the average flow rate over the sampling event (m^3 day^{-1})

Mass removals were calculated during each season throughout the duration of the experiment. Removal rates for the SF wetland prior to the summer of 2010 are not reported in this study since the influent had undergone treatment from the FWS wetland before flowing to the SF wetland (Jokerst *et al.* 2011 submitted). The removal rates of the SF wetland reported are the removal rates collected and reported after the wetlands were separated and untreated graywater was supplied to the cell.

4.3 Results and Discussion:

The constructed wetland operated over a 2 year period from September 2008 to November 2010 and a total of 47 sampling events were conducted to evaluate seasonal performance under varying HLR. Influent and effluent water quality parameters were monitored to evaluate the performance of the wetlands. The average influent graywater characteristics are described in Table 4.4.

Table 4.4 Influent Graywater Characteristics

Constituent	Mean ± S.E.
pH	6.44±0.43
Turbidity NTU	34.6 ± 16.2
SC	237.71± 49
BOD_5 (mg L^{-1})	75.56±37.2
Total Soilds (TS) (mg L^{-1})	194.65 ± 65
Total Suspended Solids (TSS) (mg L^{-1})	194.65 ±65
TOC (mg L^{-1})	38.3±17.5
Disolved Organic Carbon (DOC) (mg L^{-1})	24.5±9.4
TN (mg L^{-1})	12.5±6.3
Ammonia (mg L^{-1})	9.26±5.9

4.3.1 Hydraulic Loading Rates

Desired loading rates were established using the peristaltic pump and the amount of graywater obtained from the residence halls described in section 3.2.2. Actual HLR varied due to fluctuating flow rates from the ACB, availability of graywater at the residence halls, and the formation of ice on top of the FWS and SF wetlands in the winter months (Figure 4.6 and Figure 4.7). The Fall ‘09 and Winter ’09-’10 had higher loading rates due to an increase of activity in the ACB and higher graywater flow rates than expected. The low HLR in winter was a result of a decrease of graywater production from students leaving for winter break.

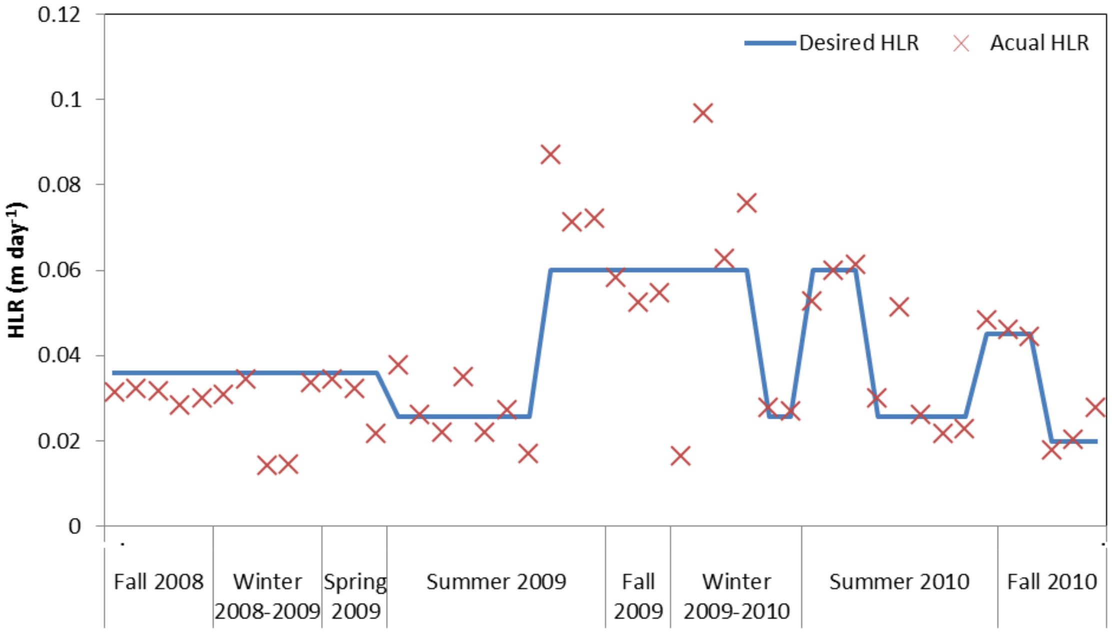

Figure 4.6 Actual and Desired HLR for the FWS Wetland

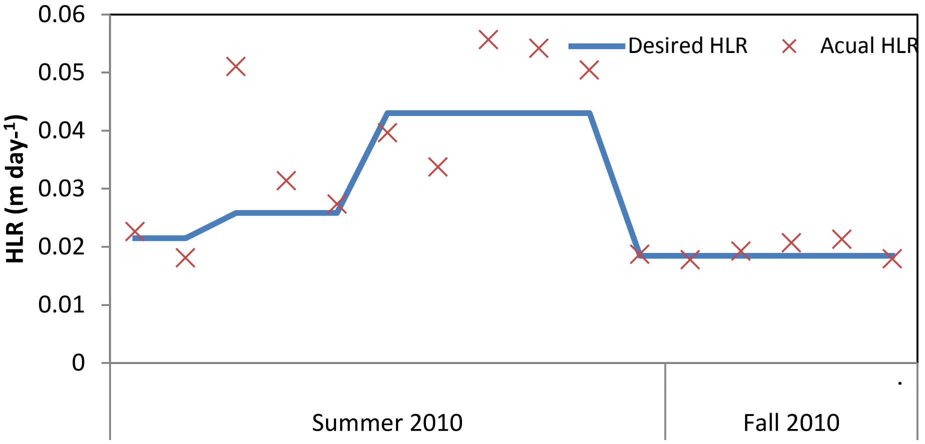

Figure 4.7 Actual and Desired HLR for the SF Wetland

4.3.2 Overall Performance of the FWS Wetland

Previous studies of constructed wetlands found that a FWS wetland substantially reduced the mass removal of graywater constituents during the fall, spring and summer seasons (Jokerst *et al.* 2011 submitted). When evaluating the FWS alone, the results from Jokerst *et al.* (2011) showed the highest removal occurred in the summer season with approximately 50% removal of BOD_5, 51% removal of TOC, 83% removal of TN, and 96% removal of ammonia.

Approximately 36 total samples were evaluated to determine the mass removal rates (12 in fall, 7 in winter, and 17 in summer, Figure 4.8). The FWS wetland showed the highest percent removal by mass for all the constituents in the summer season, followed by fall and the lowest percentage of removal occurred in winter.

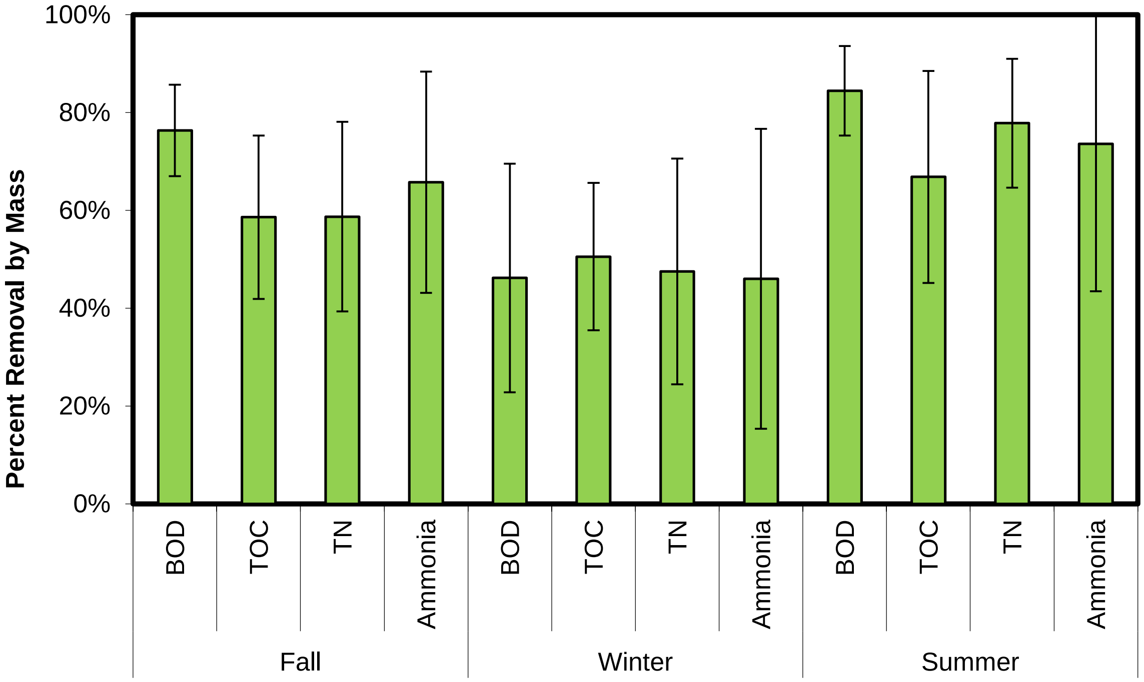

Figure 4.8. Seasonal Mass Removal for the FWS Wetland

The results from this experiment were comparable to the seasonal removal rates found in Jokerst *et al.* 2011, during fall, and summer including summer BOD_5 (84%) and TN (78%). The results within this study showed higher removals, which may be a result of increased HRT.

Summer removal rates are higher due to greater vegetation uptake, increased microbiological activity due to the higher temperatures, and higher detention times due to higher ET losses. Winter season demonstrated the lowest removals for all constituents, which was expected given that with the colder temperatures there is reduced biological activity, snow and ice accumulation prevents oxygen transfer to the wetland and there is an increase in plant nutrient contribution (plant litter) that occurrs as the plants go dormant at the end of the warm season. Spring and fall seasons were the seasons where the wetlands were transitioning from a dormant to an active stage, or vice versa.

4.3.3 HLR and Precentage of Mass Removal for the FWS Wetland

A coorilation between the HLR and percentage of mass removal was examined during this experiment. In general higher loading rates produced lower mass removal of the constituents examined (Figure 4.9 - 4.17) especially durning the winter and fall seasons. Summer BOD5 (Figure 4.11) didn't show a prevlent downword trend as seen in the in the other seasons, which may be a result of higher biological activity seen in the wetlands and limited HLR resuting from limited graywater soucres durning the summer. Higher activity in the wetland will result in a higher mass removal. An increased HLR may result in a larger downward trend. These graphs suggest that additional experiments at higher HLR are required for summer to determine the summer *k*.

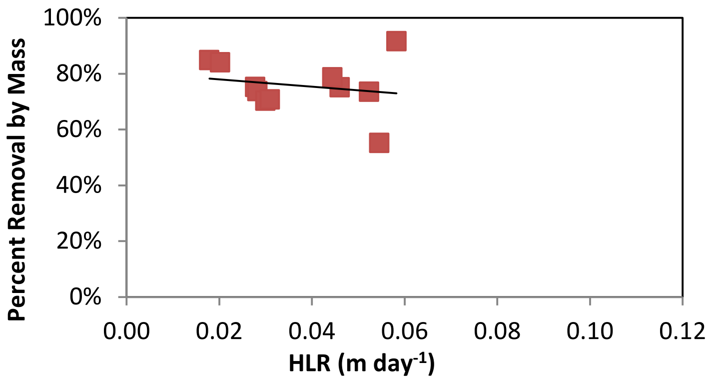

Figure 4.9. Relationship between Mass Removal and HLR for BOD$_5$ in the FWS (Fall)

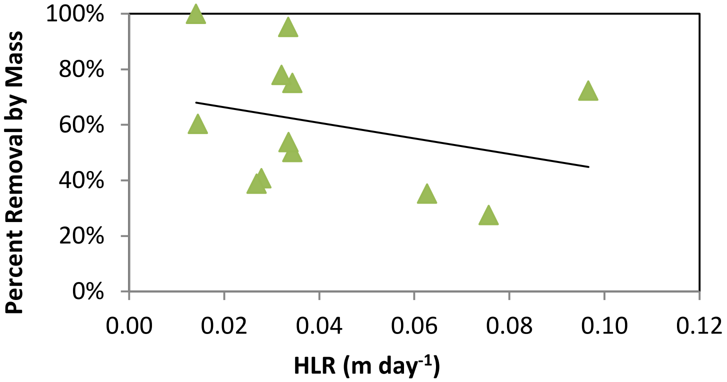

Figure 4.10. Relationship between Mass Removal and HLR for BOD$_5$ in the FWS (Winter)

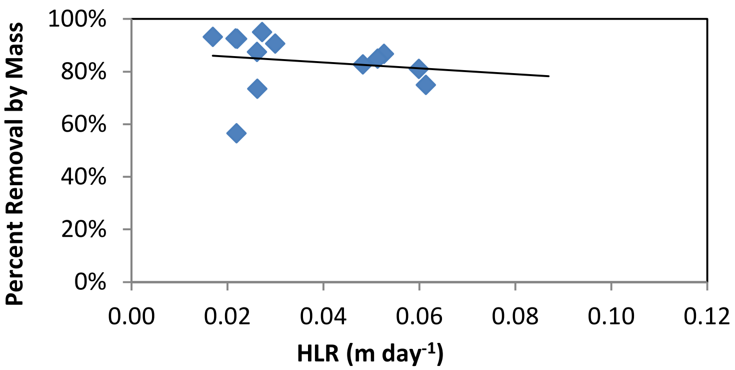

Figure 4.11. Relationship between Mass Removal and HLR for BOD$_5$ in the FWS (Summer)

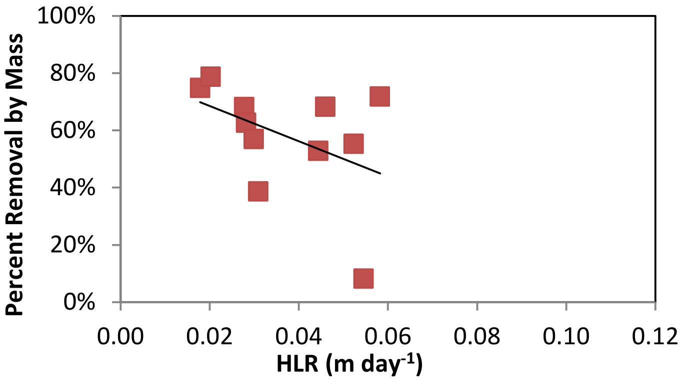

Figure 4.12. Relationship between Mass Removal and HLR for TN in the FWS (Fall)

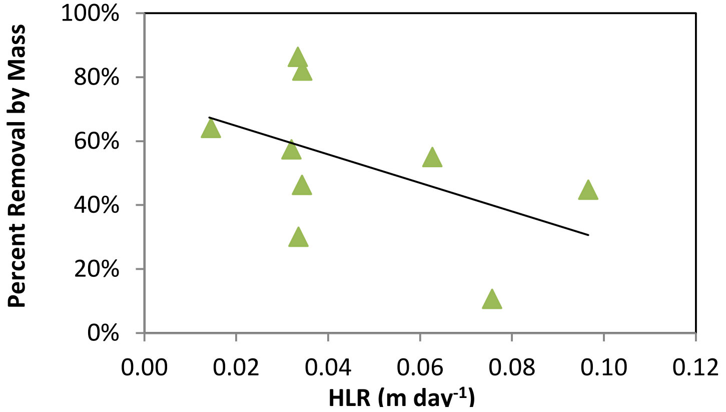

Figure 4.13. Relationship between Mass Removal and HLR for TN in the FWS (Winter)

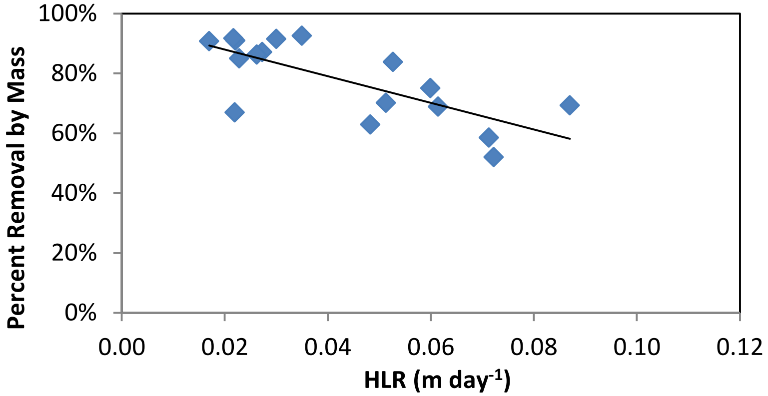

Figure 4.14. Relationship between Mass Removal and HLR for TN in the FWS (Summer)

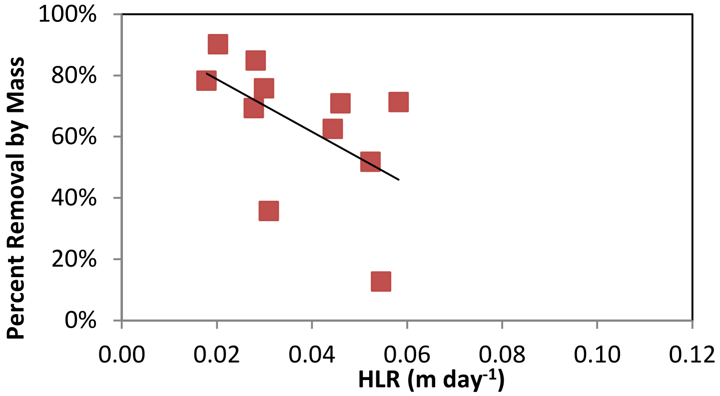

Figure 4.15. Relationship between Mass Removal and HLR for Ammonia in the FWS (Fall)

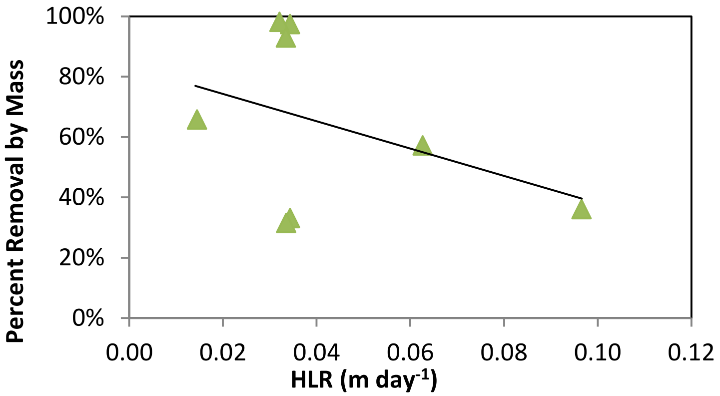

Figure 4.16. Relationship between Mass Removal and HLR for Ammonia in the FWS (Winter)

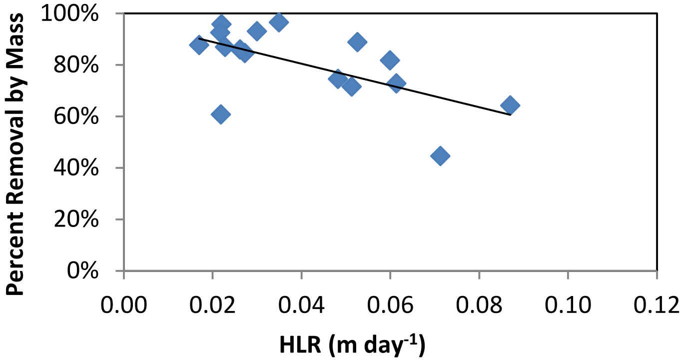

Figure 4.17. Relationship between Mass Removal and HLR for Ammonia in the FWS (Summer)

4.3.4 Parameter Estimation for the FWS Wetland

Parameter values for *k* and *C** for the FWS wetland were estimated based on Equation 4.1 (Table 4.5). Examples of plots comparing C_{eff_obs} vs. C_{eff_cal} were generated using the parameter estimations for BOD_5 removal are provided (Figure 4.18 through Figure 4.20). Additional plots for the remaining constituents (TOC, BOD_5, ammonia) and modle fit curves are provided in Appendix A for the FWS wetland and Appendix B for the SF wetland.

Table 4.5. *k* and *C Estimates for Graywater Removal in the FWS Wetland**
(reported for operation of the wetland through 2009 and 2010, unless otherwise noted).

		FWS		
		Summer	**Fall**	**Winter**
BOD_5	***k* (m yr^{-1})**	15.9	15.2	5.5
	***C** (mg L^{-1})**	6.4	101	5.01
	RSSE/n	275	103	266
TN	***k* (m yr^{-1})**	16.4	8.5	5.5
	***C** (mg L^{-1})**	11	31	31
	RSSE/n	1.2	2.6	3.6
Ammonia	***k* (m yr^{-1})**	14.7	7.4	14
	***C** mg L^{-1})**	0.2	0	31
	RSSE/n	4.7	59.5	267

[1] *C** upper/ lower bounds based on literature values

[2] report value for 2009

[3] reported value for 2010

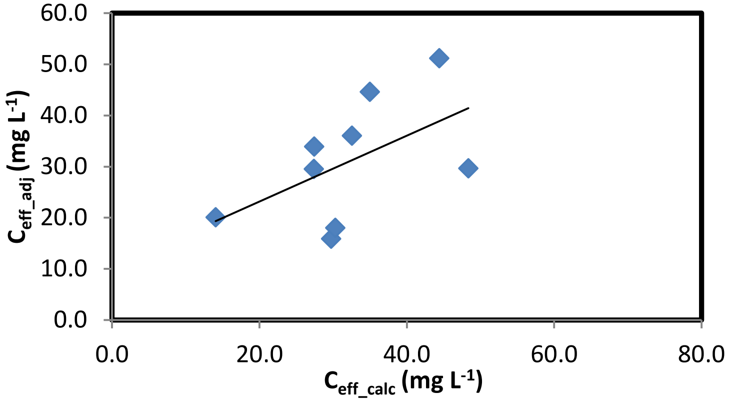

Figure 4.18. Relationship between C_{eff_cal} and C_{eff_obs} for BOD_5 in the FWS (Fall) (k = 15.2 C^* = 10 R^2=0.29)

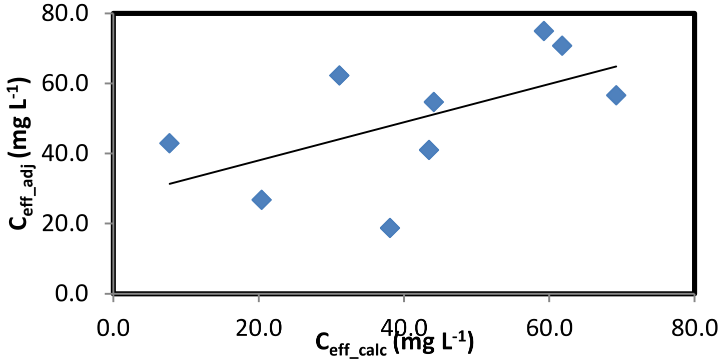

Figure 4.19. Relationship between C_{eff_cal} and C_{eff_obs} for BOD_5 in the FWS (Winter) (k = 5.5 C^* = 5 R^2=0.32)

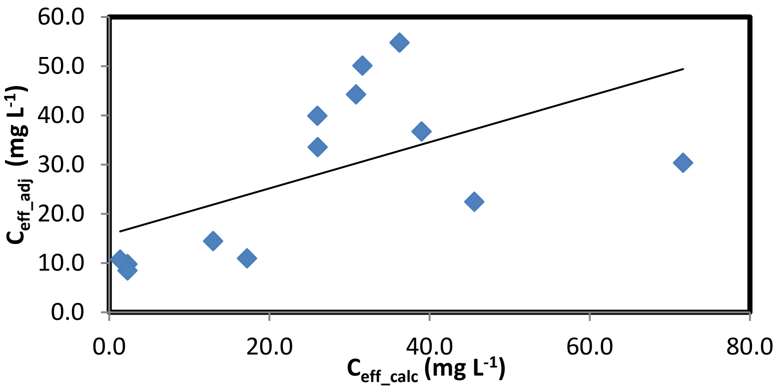

Figure 4.20 Relationship between C_{eff_cal} and C_{eff_obs} for BOD_5 in the FWS (Summer) (k = 15.9 C^* = 6.4 R^2=0.32)

RSSE/n values ranged from a high of 225 (BOD_5) and to a low of 4.7 (ammonia). The *RSSE/n* was influenced by the magnitude of the constituent concentration, which meant that the *RSSE/n* numbers for the BOD_5 values (average C_{eff_adj} = 75.6 mg L^{-1}) were expected to be higher than the ammonia values (average C_{eff_adj} = 9.3 mg L^{-1}) because the average influent concentration of BOD_5 was significantly larger than the concentration of ammonia. While the parameter estimation procedure was successful, and trends were observed between $C_{eff\_ adj}$ and HLR, some *RSSE/n* values were slightly high. This is expected since the wetland system was located outdoors, subject to varying climatic conditions and also varying loads of graywater constituents (Table 4.1).

In general, estimates for *k* were lower and *C** were larger in winter compared to other seasons (Table 4.5). The smaller *k* values are most likely linked to the colder temperatures and slower microbiological activity. Greatest removal rates were observed for all of the constituents in the summer months, consistent with higher mass removal rates (Figure 4.6). The high *k* over summer may be attributed to the warmer temperatures (Table 4.5) resulting in increased microbiological activity and mature plant growth.

Trends between $C_{eff\_ adj}$ and HLR for TOC were not observed during the fall and winter seasons (Figure 4.21). This may be the result of additional falling litter and leaching of organics from dead or dying plant biomass into the wetland during these seasons. The additional organic matter can contribute to a high *C**, where *C** dictates C_{eff_adj} more than removal of constituents through biological activity and plant uptake. Summer TOC data appeared to fit reasonably well (Appendix B).

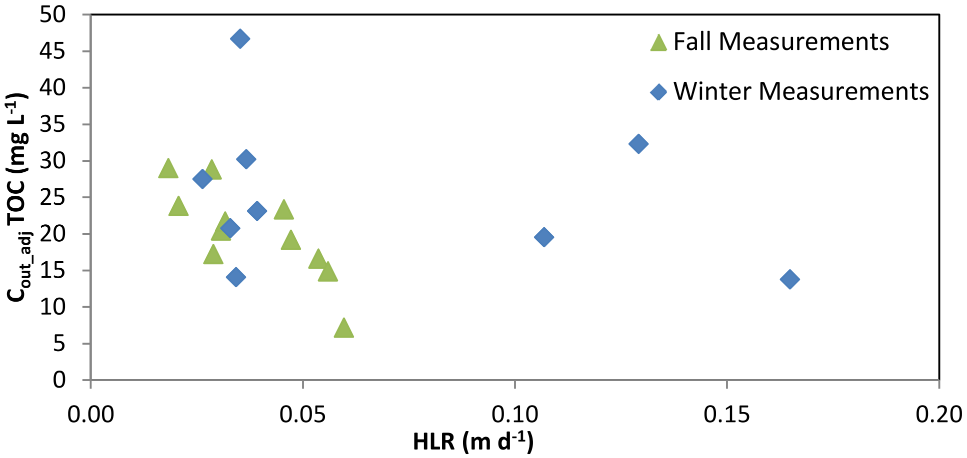

Figure 4.21. Fall and Winter TOC Measurements

Comparing parameter estimates for BOD_5, TN and ammonia with the values for these parameters available in the literature (Tables 4.6 - 4.8), it is evident that the wetland performance for graywater treatment is different than the previously applied design criteria for domestic wastewater and agricultural wastewater wetlands. TOC values were not compared because the results determined from this study were inconclusive and values for TOC in the literature could not be found.

When comparing our results with the results reported in the literature, some constituent removal rates were comparable where as other constituents varied greatly from the removal rates fround in the literature. Our parameter estimates showed that the *k* for BOD_5 in graywater were smaller than the reported *k* from other constructed wetland treated wastewater sources including municipal and livestock effluent wastewater. Our reported winter BOD_5 *k* values are even lower than the cold climate conditions that Jamieson et al (2007) studied. The TN removal rates were consistent with values for livestock effluent, but lower than those reported for municipal wastewater (Table 4.7). Removal rates for ammonia were comparable to the livestock effluent wastewater, and higher than the values given for the cold climate conditions that Jamieson et al (2007) studied. However, the *k* values for ammonia were not as high as removal rates observed for municipal wastewater.

Table 4.6. BOD_5 Literature *k* and *C for FWS Wetlands**

k (m yr^{-1})	Range for *k* (m yr^{-1})	*C** (mg L^{-1})	Influent Concentration (mg L^{-1})	Reference	Type of Wastewater
15.9	N/A	6.4	74.5	Current Study	graywater (summer)
5.6	N/A	5	99.9	Current Study	graywater (winter)
34 (±22)	N/A	6.2 (±3.5)	N/A	(Kadlec and Knight 1996)	municipal wastewater
22	7 - 68	N/A	N/A	Knight *et al.*, 2000	livestock wastewater
7.0	38.7 - 0.6	N/A	1747 (±860)	Jamieson *et al.*, 2007	livestock wastewater (in cold climates)

N/A = Not Available

Table 4.7. TN Literature *k* and *C Information for FWS Wetlands**

k (m yr^{-1})	*Range k* (m yr^{-1})	*C** (mg L^{-1})	Influent Concentrations (mg L^{-1})	Reference	Type of Wastewater
16.4	N/A	1	14.6	Current Study	graywater (summer)
5.5	N/A	3	17.3	Current Study	graywater (winter)
15.3	N/A	1.5	>2	(Kadlec and Knight 1996)	municipal wastewater
14	5-32	NA	NA	Knight *et al.*, 2000	livestock wastewater

N/A = Not Available

Table 4.8. Ammonia Literature *k* and *C Information for FWS Wetlands**

k (m yr^{-1})	*Range k* (m yr^{-1})	*C** (mg L^{-1})	Influent Concentration (mg L^{-1})	Reference	Type of Wastewater
3.9 - 14.4	N/A	0-3	3.9-14.4	Current Study	graywater (summer)
14	N/A	3	11.7	Current Study	graywater winter
18	N/A	N/A	N/A	(Kadlec and Knight 1996)	municipal wastewater
10	-1. to 26	3	N/A	Knight *et al.* 2000	livestock wastewater
4.3	-0.3 to 17.2	N/A	188 (±131)	Jamieson *et al.* 2007	livestock wastewater (in cold climates)

N/A = Not Available

Lower observed *k* for BOD$_5$ for graywater compared to domestic wastewater may be attributed to the lower load of organics (Table 4.9). Biological removal rates of surfactants have been found to be slower than typical organic material in domestic wastewater (Rittmann 2001; Sharvelle et al. 2007). In addition, the C:N ratio in graywater is lower in graywater (150:1) compared to domestic wastewater (8:1). Nitrogen may limit the rate at which carbon is biologically removed (through microorganisms and plant uptake) in graywater. This may also explain the relatively high *k* values observed for TN and ammonia in graywater compared to domestic and livestock wastewaters (Table 4.7-Table 4.8).

Table 4.9. Composition of Graywater to Domestic Wastewater

	Graywater Range[1] (mg L^{-1})	Domestic Wastewater Range[2] (mg L^{-1})
Chemical Oxygen Demand (COD)	77-240	250-1000
BOD_5	26-130	110-400
TSS	7-207	100-350
TN	0.36-0.64	20-85
Total Phosphorus (TP)	0.28-0.779	4-15
Total Coliform (CFU/100mL)	6.0×10^3-3.2×10^5	10^6-10^9
E. Coli (CFU/100mL)	<100-2800	

[1](Eriksson et al. 2002)
[2](Tchobanoglous, et. al, 2004)

4.3.5 Design Application for the FWS Wetland

BOD$_{5.}$ Parameter estimates for BOD_5 *k* and *C** for graywater treatment in a constructed wetland were applied to estimate the surface area needed for a constructed wetland compared to the application of previously available *k* and *C** values which would be used for sizing of a graywater constructed wetland in the absence of this study. The influent values that were used for this comparison were 100 mg L^{-1} (based on the average BOD_5 values reported in this study), a flow of 250 m^3 day^{-1}, and a *C** of 6.4. The comparison (Table 4.10) determined the required SA using the *k* (summer and winter) values reported in this study compared to the methods provided by: 1) the EPA manual for municipal wastewater using a ALR of 40 (EPA, 2000); 2) using the *k* reported for municipal wastewater (Kadlec and Knight 1996); and 3) the *k* values reported for livestock wastewater (Knight *et al.*, 2000). A desired effluent BOD concentration of 20 mg L^{-1} was applied based on the reuse effluent standard recommended by EPA (EPA 2000).

Table 4.10. Example Area Requirement for a Constructed Wetland for Graywater Treatment Based on Current Study and Previously Available Methods

	Graywater (Summer 2009)[1]	**Graywater (Winter)[1]**	**Dairy Effluent[2]**	**Municipal Wastewater[3]**	**Municipal Wastewater[4]**
k **(m yr^{-1})**	15.9	5.6	22	34	N/A
Loading Rate (kg ha^{-1} d^{-1}) (10^6 mg kg^{-1})	N/A	N/A	N/A	N/A	40
SA (m^2)	11,000	31,400	8,000	5,200	6,250

[1]Current Study. [2] Knight *et al.*, 2000, [3]Kadlec and Knight 1996, [4] (EPA 2000)
N/A = Not Available

It was expected that the EPA method for determining the required SA would require be larger when compared to the evaluated *k* rate from this experiment or from the literature values because the EPA values include additional safety factors and a broad approach to wetland treatment regulations. However, the EPA calculations were only slightly larger when compared to the other municipal wastewater *k* presented by Kadlec and Knight (1996) and smaller than the calcuation using the *k* rates detemined from this experiment and from the dairy effluent calculation (Knight, 2000).

The *k* determined from this experiment are lower that other established *k* (Kadlec and Knight, 1996, Knight 2000), because of the differences in the types of wastewater treated. Other literature values are based on wastewater that may have a larger amount of degradable organic constituents when compared to graywater, which provides for a higher removal rate and therefore a smaller surface area.

Since the *k* values were higher for the livestock and municipal wastewater it would be expected that the required SA would be less than the values obtained from this study. As shown in Table 4.10 the smallest *k* rate which would result in the largest required SA was generated from the winter graywater data. The differences between the the *k* rates using the numbers from this experiment and the numbers from the EPA, or Kadlec and Knight (1996), demonstrate that the current guidelines based on wastewater *k* rates or EPA guildlines would have underestimated the required size of a constructed wetland that is needed to treat graywater.

The values between the summer and winter removal rates are significantly different. Given the values from the winter removal rates the required SA needed for a constructed wetland treating graywater in the winter would need to be approximately 3 times the size of the summer wetland (Table 4.10). When designing a wetland for treatment all year (i.e. for a toilet flushing application), the land required will be significantly more when compared to seasonal treatment of graywater (i.e. for summer irrigation).

The parameters *k* and *C** will vary between wetlands because of differences in climate, temperature, loading rates, influent concentrations, and the type of wastewater etc. A good example of this is the differences seen between the removal rates within this study vs. other *k* rates due to the influent concentration values. When easily degradable organics enter a wetland the organics will be removed at a higher rate. After those easily degradable organics are removed the tougher organic constituents that remain will slow the removal rate.

TN. In addition to the BOD_5 as a desing criteria, a comparison between the different sizing criteria was evaluated using the existing TN values available in the literature with the values determined from this experiment (Table 4.11). The influent values used for this comparison were determined to be TN of 15 mg L^{-1} (based on the average TN values reported in the summer season), a flow of 250 m^3 day^{-1}, and a C^* of 1. The comparison (Table 4.11) determined the required SA using the k (summer and winter) values reported in this study compared to: 1) the k values reported from municipal wastewater (Kadlec and Knight 1996); and 2) the k values reported from livestock wastewater (Knight *et al.* 2000). The methods are evaluted to determine the required SA for treatment to an effluent concentration of 5 mg L^{-1}.

Table 4.11. Example Area Requirement for a Constructed Wetland for Graywater Treatment Based on Current Study and Previously Available Methods

	Graywater (Summer)[1]	**Graywater (Winter)[1]**	**Dairy Effluent[2]**	**Municipal Wastewater[3]**
***k* (m yr^{-1})**	16.4	5.5	14	15.3
SA (m^2)	6,500	20,000	7,700	7,000

[1]Current Study.
[2] (Knight *et al.* 2000),
[3]Kadlec and Knight 1996,

Since the determined k values for this study were slightly higher than those determined for livestock and municipal wastewater it would be expected that the required SA would be smaller. As shown in Table 4.11 the largest required SA was the winter graywater data because it had the lowest k value. Had a wetland been designed based on TN values found in the literature, the SA would have been accurate for the summer removal rates but the SA of the wetland during the winter months would have been significantly underestimated compared to the data from the current study.

The values determined for summer and winter removal rates show a significant difference in the SA needed, which equates to a required SA of the winter being approximately 3 times the size of the summer wetland (Table 4.11). When designing a wetland for treatment all year (i.e. for a toilet flushing application), the land required will be significantly larger when compared to seasonal treatment durning the summer season (i.e. for irrigation application).

4.3.6 Overall Performance of the SF Wetland

Approximately 15 total samples were evaluated to determine the mass removal rates of the SF wetland (5 in fall, and 11 in summer). Mass removal observed in the SF wetland showed the highest removal rates of the constituents in the summer season, followed by the fall season (Figure 4.20). There were no literature values found on the mass removal of graywater through a SF wetland to compare to this study.

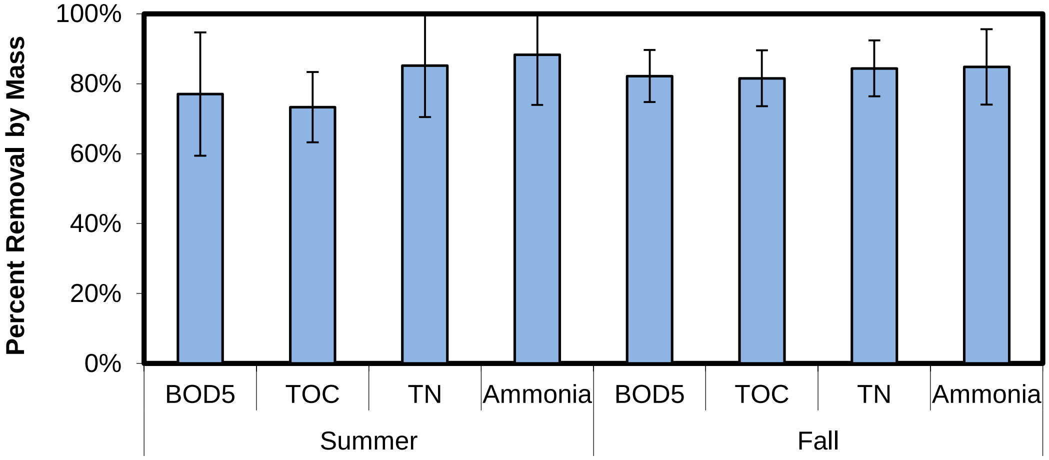

Figure 4.22 Seasonal Mass Removal for the SF Wetland.

4.3.7 HLR and Precentage of Mass Removal for the SF Wetland

A coorilation between the HLR and percentage of mass removal was examined for the SF wetland. Summer BOD_5, TN, and ammonia showed downword trend as HLR increased (Figure 4.23 – 4.25).

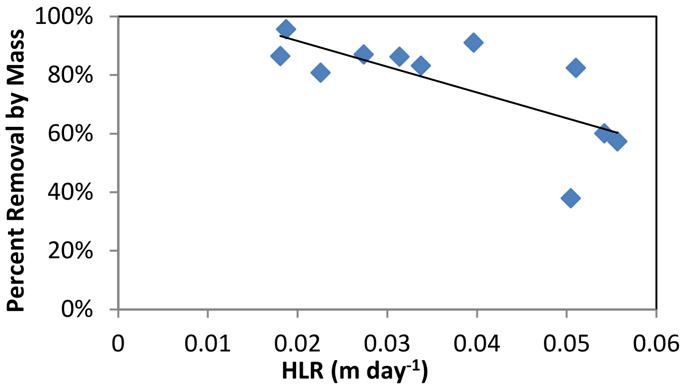

Figure 4.23. Relationship between Mass Removal and HLR for BOD$_5$ in the SF (Summer)

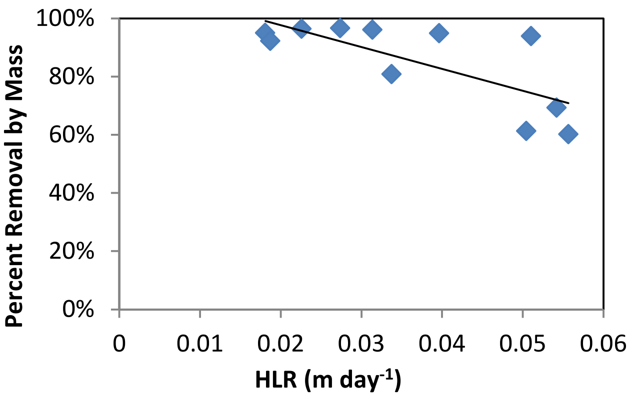

Figure 4.24. Relationship between Mass Removal and HLR for TN in the SF (Summer)

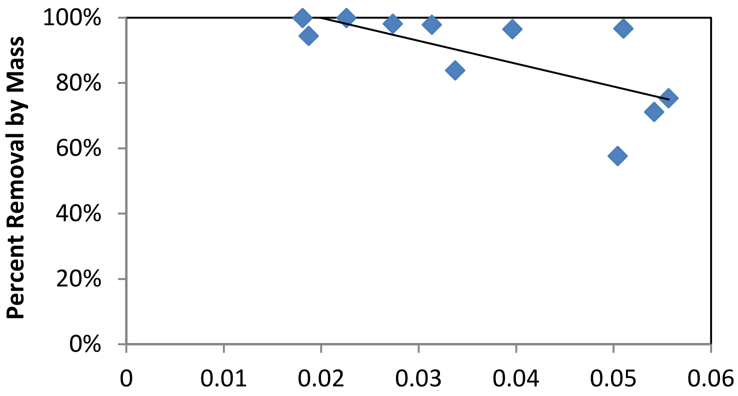

Figure 4.25. Relationship between Mass Removal and HLR for Ammonia in the FWS (Summer)

4.3.8 Parameter Estimation for the SF Wetland

Parameter values for the *k* and *C** for the SF wetland were estimated based on Equation 4.1 (Table 4.12). Examples of plots generated from parameter estimations for BOD_5 removal are provided (Figure 4.26). Additional plots for the other constituents (TOC, BOD_5, and TN) are provided in Appendix B.

Table 4.12. *k* and *C Estimates for Graywater Removal in the SF Wetland** (Reported for operation of the wetland in the summer of 2010).

		SF Summer
BOD_5	***k* (m yr^{-1})**	19.1
	***C** (mg L^{-1})**	8
	RSSE/n	188
TOC	***k* (m yr^{-1})**	22.8
	***C** (mg L^{-1})**	8
	RSSE/n	43.4
TN	***k* (m yr^{-1})**	21.3
	***C** (mg L^{-1})**	0.8
	RSSE/n	3.3
Ammonia	***k* (m yr^{-1})**	32.6
	***C** mg L^{-1})**	1.6
	RSSE/n	4.9

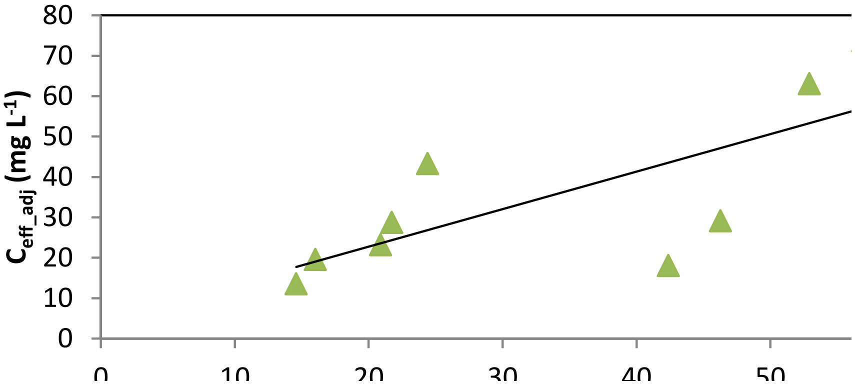

Figure 4.26. Relationship between Ceff_cal and Ceff_obs for BOD_5 in the SF (Summer) (k = 19.1 C^* = 8 R^2=0.53)

The SF wetland summer data was analyzed by comparing this experiment's BOD_5, TN, and ammonia k parameters with other known design methods (Table 4.13, through Table 4.15). Only the summer values were compared because the values for the other seasons were inconclusive. During the comparison it was evident that wetland treatment for graywater removal is different than the design criteria based on current literature.

Table 4.13. BOD_5 Literature and k and C^* Information for SF Wetlands

k (m yr^{-1})	C^* (mg L^{-1})	Influent Concentration (mg L^{-1})	Reference	Type of Wastewater
19.1	8	98.3	Current Study	graywater (summer)
180 (±61)	9.8(±5.6)	N/A	(Kadlec and Knight 1996)	municipal wastewater

N/A = Not Available

Table 4.14. TN Literature and *k* and *C Information for SF Wetlands**

k (m yr^{-1})	*C** (mg L^{-1})	Influent Concentrations (mg L^{-1})	Reference	Type of Wastewater
21.3	0.8	14.9	Current Study	graywater (summer)
15.1	1.5	>2	(Kadlec and Knight 1996)	municipal wastewater
23.6	19.7	67(±25.9)	(Tanner *et al.* 1998)	livestock wastewater

Table 4.15. Ammonia Literature and *k* and *C Information for SF Wetlands**

k (m yr^{-1})	*C** (mg L^{-1})	Influent Concentration (mg L^{-1})	Reference	Type of Wastewater
32.6	1.6	14.2	Current Study	graywater (summer)
34	N/A	N/A	(Kadlec and Knight 1996)	municipal wastewater
16.1	14.4	40(±21)	(Tanner *et al.* 1998)	livestock wastewater

When comparing our results with the results reported in the literature some constituent removal rates were comparable where as other constituents did not match the removal rates from the literature. Our parameter estimates showed that the removal rates of BOD_5 in graywater were smaller than those determined for constructed wetlands used to treat municipal wastewater sources (Table 4.13). The TN removal rates were mid-range of the literature values. This study's *k* were higher than the literature values for municipal wastewater and lower than the literature values for the livestock effluent. Ammonia *k* values were also mid-range of the literature values but were closer to the municipal effluent and double the removal reported for livestock wastewater.

4.3.9 Design Application for the SF Wetland

BOD_5 Analysis. The BOD_5 k and C^* values from this experiment were compared with the existing values set forth in the literature to analyze the SA needed for SF constructed wetlands that treats graywater (Table 4.13). The influent values that were used for this comparison were determined to be BOD_5 of 100 mg L^{-1} (based on the average BOD_5 value reported in this study), a flow of 250 m^3 day^{-1}, and a C^* of 8. The comparison (Table 4.16) determined the required SA using the k (summer and winter) values reported in this study compared to the method provided by 1) the EPA manual for municipal wastewater (EPA, 2000), and 2) the k reported form municipal wastewater (Kadlec and Knight 1996). The BOD_5 influent concentration for graywater is being compared to the same standard of 20 mg L^{-1} effluent based on the reuse effluent standard required for domestic wastewater (EPA 2000).

Table 4.16. Example Area Requirement for a Constructed Wetland for Graywater Treatment Based on Current Study and Previously Avaiable Methods

	Graywater (Summer)[1]	**Municipal Wastewater[2]**	**Municipal Wastewater[3]**
k ($m\ yr^{-1}$)	19.1	180	N/A
Loading Rate ($g\ m^{-2}\ d^{-1} 10^6\ mg\ kg^{-1}$)	N/A	N/A	6
SA (m^2)	9,700	1,000	4200

[1]Current Study. [2] Kadlec and Knight 1996, [3] (EPA 2000)
N/A = Not Available

The k determined for this experiment was less than other literature values for k. When determining the size of a constructed wetland the smaller k would require a lager SA for treatment. The samller k value will result in a larger required SA needed for graywater treatment. The differences between the literature k values and this study's k values demonstrate that the current guidelines, based on municipal wastewater k rates, would have underestimated the required size of a SF constructed wetland used to treat graywater.

TN Analysis –A comparison between the different sizing criteria was evaluated using the existing methods TN values available in the literature with the values determined from this experiment (Table 4.17). The influent values used for this comparison were determined to be TN of 14 mg L^{-1} (based on the average TN value reported in the summer season), a flow of 250 m^3 day^{-1}, and a C^* of 1. The comparison (Table 4.17) determined the required SA using the *k* (summer) value reported in this study compared to values from 1) the *k* reported from municipal wastewater (Kadlec and Knight 1996); and 2) the *k* value reported for livestock wastewater (Tanner *et al.* 1998). The methods are evaluted to determine the required SA for treatment to an effluent concentration of 3 mg L^{-1}.

Table 4.17. Example Area Requirement for a Constructed Wetland for Graywater Treatment Based on Current Study and Previously Avaiable Methods

	Graywater (Summer)[1]	**Dairy Effluent[2]**	**Municipal Wastewater[3]**
***k* (m yr^{-1})**	21.3	23.6	15.1
SA (m^2)	8,000	7,200	11,300

[1]Current Study. [2]
Tanner *et al.* 1998,
[3]Kadlec and Knight 1996

Results show that the *k* rate and associated SA of a wetland is similar for graywater and dairy effluent, but smaller than the municipal wastewater calculation. As shown in Table 4.17 the largest required SA was the municipal wastewater calculation because it had the lowest *k* value. The differences demonstrate that the current guidelines based on wastewater *k* rates for municipal wastewater would have overestimated the required size of a constructed wetland that is used to treat graywater.

When comparing the results obtained in this study between the FWS and SF wetland (summer season), it appears that the SF wetland requires a smaller square area for both BOD_5 and TN values (Table 4.16 and Table 4.17).

4.4 Conclusion

Wetlands can offer a low cost, low maintenance treatment system for improving water quality in a variety of wastewater sources. Overall performance of this study showed that using a constructed wetland for treatment of graywater is a viable method for removing graywater constituents especially during the summer months. Mass removal results were consistently higher in the summer and the summer *k* were also typically higher for BOD_5 in both the FWS and SF wetland. It is important to understand these *k* values are specific for the type of the wastewater that is being treated. Current design criteria for designing a constructed wetland are based on municipal or agricultural wastewater, which are not applicable to treatment of lower strength graywater. This study evaluated a FWS and SF wetland to determine the seasonal *k –C** method for evaluating the constructed wetland treatment of graywater.

The computed *k–C** from this study were lower than the literature values for municipal and agricultural wastewater sources. This can be attributed to the lower concentrations of organic material and nutrients that are found in graywater. The design of a wetland treating graywater with a lower HLR (higher HRT) will have a greater amount of removal. Wetland sizing based on the determined *k* and *C** values showed that use of previously available values for domestic and livestock wastewater would result in substantially under sizing of the system. Estimates of *k* and *C** provided by this study can be applied for future designs of FWS and SF wetlands for graywater treatment.

The seasonal removal rates should be considered for sizing a constructed wetland. If the wetland needs to achieve a minimum level of treatment all year and you are in a climate that includes a winter season then the winter removal rate should be considered for the design of the wetland. If irrigation is the intended use of the treated graywater then the summer removal rates can be used to design the SA of the wetland.

CHAPTER 5

SUMMARY

Water conservation efforts, including water reuse, are necessary to sustain population growth, agricultural needs, and recreational and industrial uses. Graywater reuse for non-potable demands is gaining popularity because it allows the reuse of minimally contaminated wash water generated at the home/office for non-potable demands thus reducing the demand for treated water and preserving freshwater sources. Reusing graywater can have several benefits including an overall reduction in potable water demand, reduction in wastewater production, and providing nutrients and water to plants. Graywater reuse systems are versatile and can be integrated into a single family home, large residential buildings, or anything in between.

This research was to provide guidance for individual home or business owners, regulatory agencies, or anyone who is interested in more information on graywater reuse. Although graywater has many beneficial uses, graywater reuse is not appropriate for every home or business owner. Installation of graywater reuse systems requires plumbing knowledge, investment in new equipment and materials, and possibly some additional construction costs to retrofit homes or offices. Another consideration is the legality of graywater reuse since not all states allow graywater reuse. Precautions and best management practices must be followed in order for graywater reuse to remain safe and effective. Proper installation and use of these graywater reuse operating systems are necessary and those who install and operate these systems are accountable for following these recommendations in order to safely reuse graywater. Best management practices include installation of a dual plumbing system with overflow lines to the sewer, proper treatment for the end use, ensuring proper handling of the graywater, and understanding that what flows into a graywater reuse system can adversely affect human health, plants, and the environment.

From a global perspective, constructed wetlands are gaining popularity as a cost effective, environmental conscious method for wastewater management for both developed and undeveloped countries. Constructed wetlands can offer a scalable, economically sound, low tech and easily maintained method for treating graywater on a community scale. Since there is a lower load of contaminants in graywater compared to domestic wastewater constructed wetland can provide effective treatment for graywater reuse. Current design criteria for designing a constructed wetland are based on municipal or agricultural wastewater, which are not applicable to treatment of lower strength graywater.

Results from this study showed that during the summer the mass removal of constitutents were consistently higher than the mass removal during the other seasons and the summer *k* values were typically higher both the FWS and SF wetlands than the other seasons (Table 5.1). It is important to understand these *k* values are specific for the type of the wastewater that is being treated. This study evaluated a FWS and SF wetland to determine the seasonal *k* –*C** method for evaluating the constructed wetland treatment of graywater.

Table 5.1. Composition of Graywater to Domestic Wastewater.

	FWS			SW
	Summer k (m yr^{-1})	Fall k (m yr^{-1})	Winter k (m yr^{-1})	Summer k (m yr^{-1})
BOD_5	15.9	15.2	5.6	19.1
TOC	N/A	N/A	N/A	22.8
TN	16.4	8.5	5.5	21.3
Ammonia	30.4-25.5	7.42	14	32.6

Determining removal rates was important in order to create wetland design standards for graywater treatment and reuse. In the comparison between this study's *k* rates and other design methods including the EPA standards, there was a large discrepancy in the required SA of a constructed wetland using the different methods (Chapter 4). Wetland sizing based on the BOD_5 *k* and *C** values or the EPA standards showed that use of previously available values for domestic and livestock wastewater would result in substantially under sizing the wetland for graywater treatment. The different in wetland size can be attributed to lower concentration of biodegradable organic material and nutrients that are found in graywater. Estimates of *k* and *C** provided by this study can be applied for future designs of FWS and SF wetlands for graywater treatment.

The seasonal removal rate should be considered for sizing of a wetland. If the wetlands need to achieve a minimum level of treatment all year and you are in a climate that includes a winter season then the winter removal rate should be considered for the design of the wetland. If irrigation is the intended use of the treated graywater then the summer removal rates can be used to design the surface area of the wetland.

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APPENDIX A:

SEASONAL REMOVAL RATES PLOTS FOR THE FWS CONSTRUCTED WETLAND

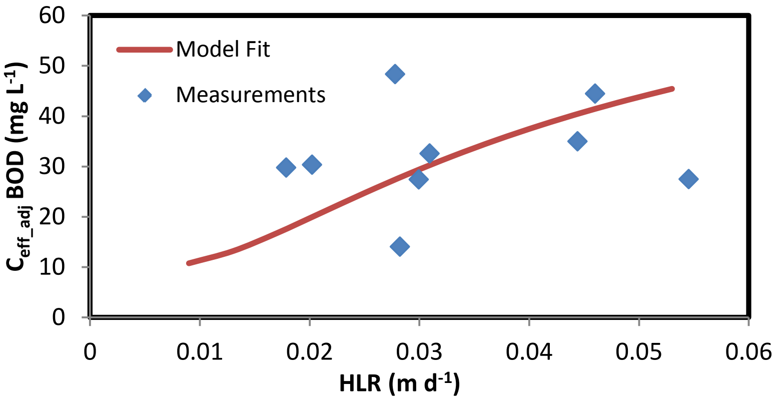

Figure A.1. Parameter Estimation for BOD$_5$ Removal in the FWS (Fall) (k = 15.2 C^* = 10 $RSSE/n$ = 103)

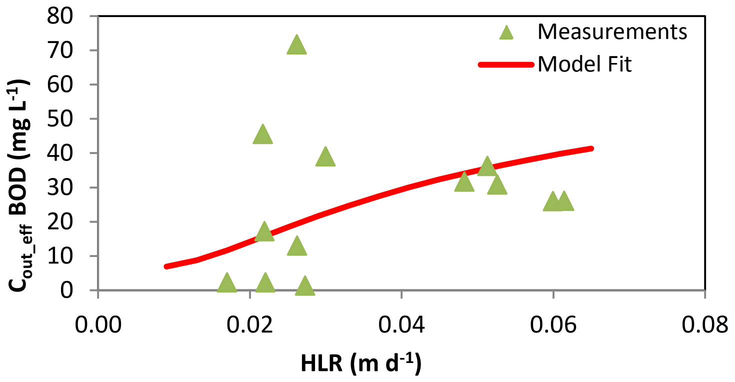

Figure A.2. Parameter Estimation for BOD$_5$ Removal in the FWS (Summer) (k =15.9 C^* = 6.4 $RSSE/n$ = 275)

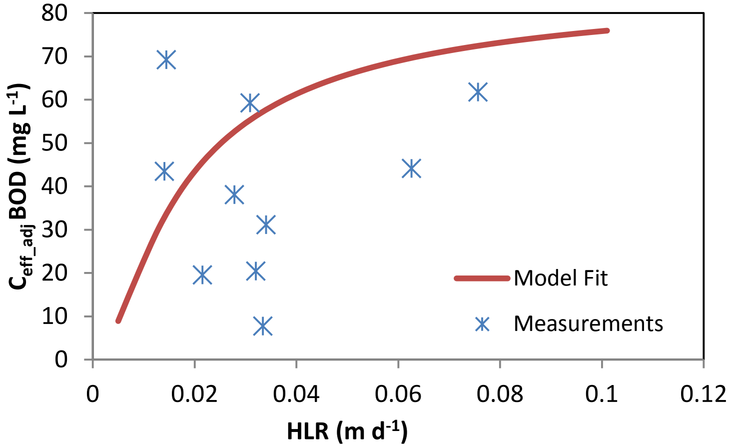

Figure A.3. Parameter Estimation for BOD$_5$ Removal in the FWS (Winter) (k = 5.6 C^* = 5 $RSSE/n$ = 266)

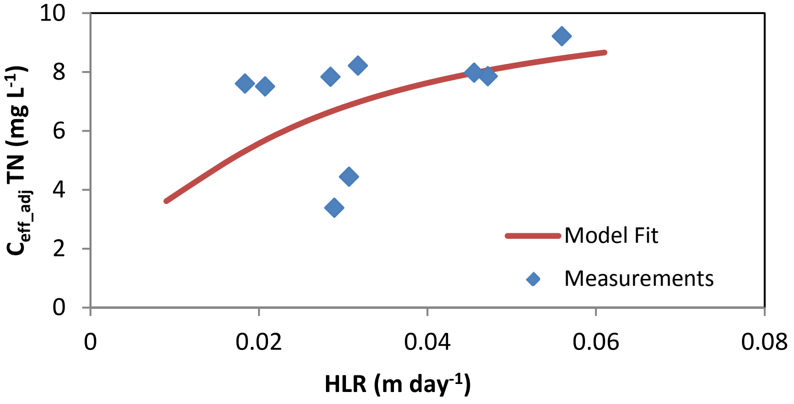

Figure A.4. Parameter Estimation for TN Removal in the FWS (Fall) (*k* = 8.5, *C** = 3, *RSSE/n* = 3)

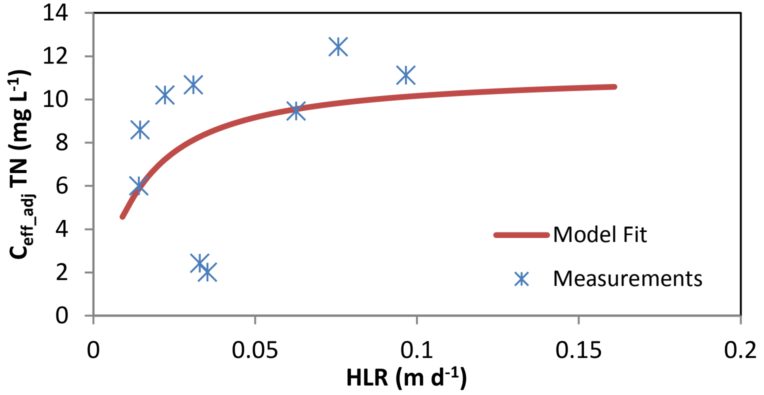

Figure A.5. Parameter Estimation for TN Removal in the FWS (Winter)(*k* = 5.5, *C** = 3, *RSSE/n* = 3.6)

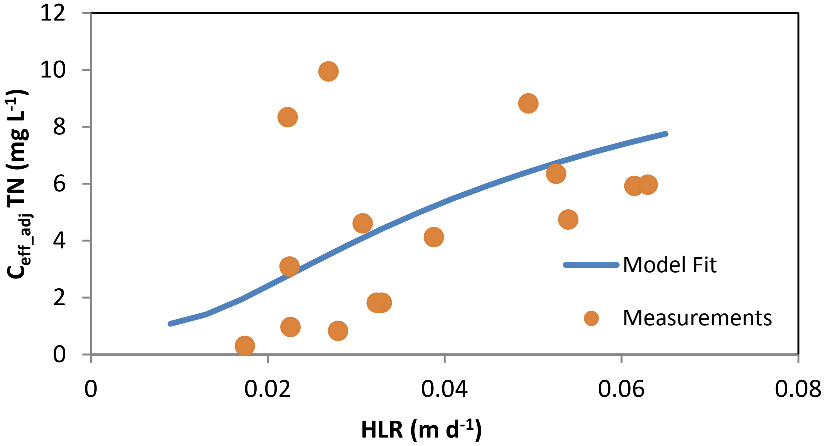

Figure A.6. Parameter Estimation for TN Removal in the FWS (Summer) (*k* = 16.4, *C** = 1, *RSSE/n* = 1.2)

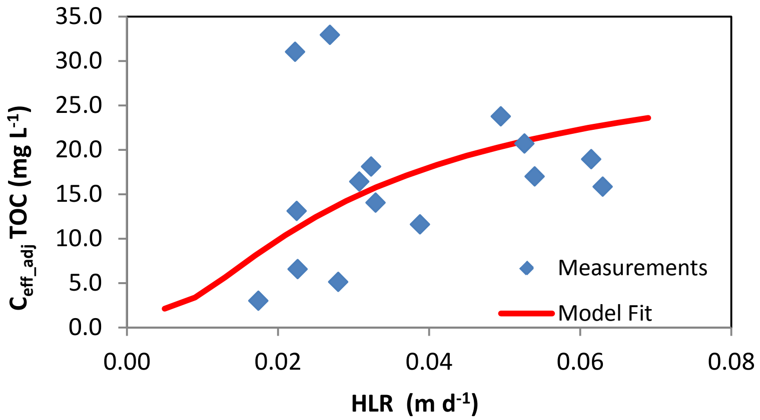

Figure A.7. Parameter Estimation for TOC Removal in the FWS (Summer) (*k* = 10.4, *C** = 2, *RSSE/n* = 24.7)

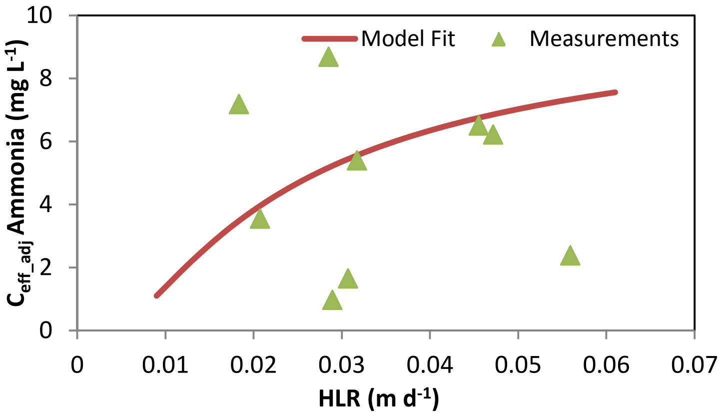

Figure A.8. Parameter Estimation for Ammonia Removal in the FWS (Fall) (k = 7.4, $C*$ = 0, $RSSE/n$ = 59.5)

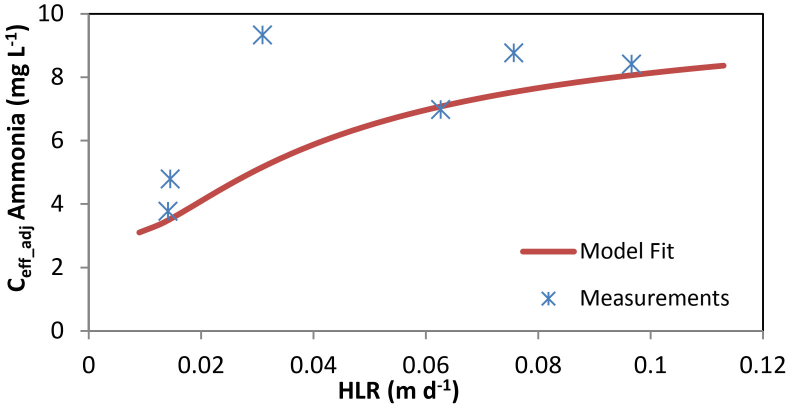

Figure A.9. Parameter Estimation for Ammonia Removal in the FWS (Winter) (k = 14.1, $C*$ = 3, $RSSE/n$ = 267)

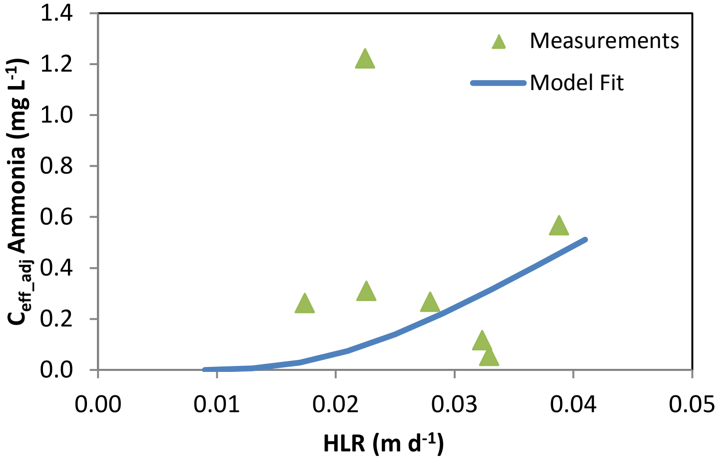

Figure A.10. Parameter Estimation for Ammonia Removal in the FWS (Summer2009) (k = 30.4, $C*$ = 0, RSSE/n = 0.2)

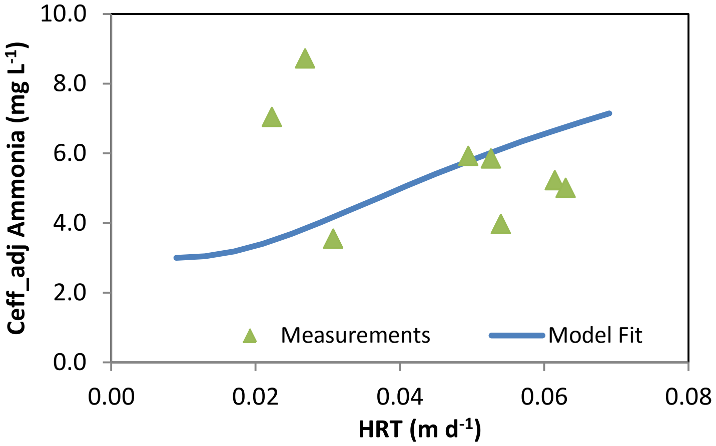

Figure A.11. Parameter Estimation for Ammonia Removal in the FWS (Summer 2010) (k = 25.5, $C*$ = 3, $RSSE/n$ = 6.1)

APPENDIX B:

SEASONAL REMOVAL RATES PLOTS FOR THE SF CONSTRUCTED WETLAND

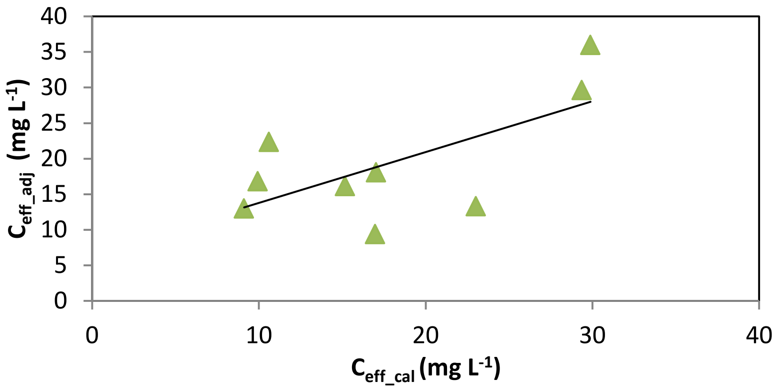

Figure B.1 Relationship between Ceff_cal and Ceff_obs for TOC in the SF (Summer) (*k* = 22.8 *C = 8 R^2=0.44)**

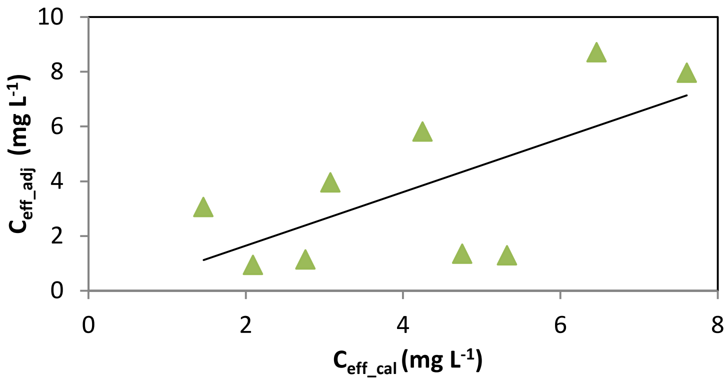

Figure B.2. Relationship between Ceff_cal and Ceff_obs for TN in the SF (Summer) (*k* = 21.3 *C = 0.8 R^2=0.44)**

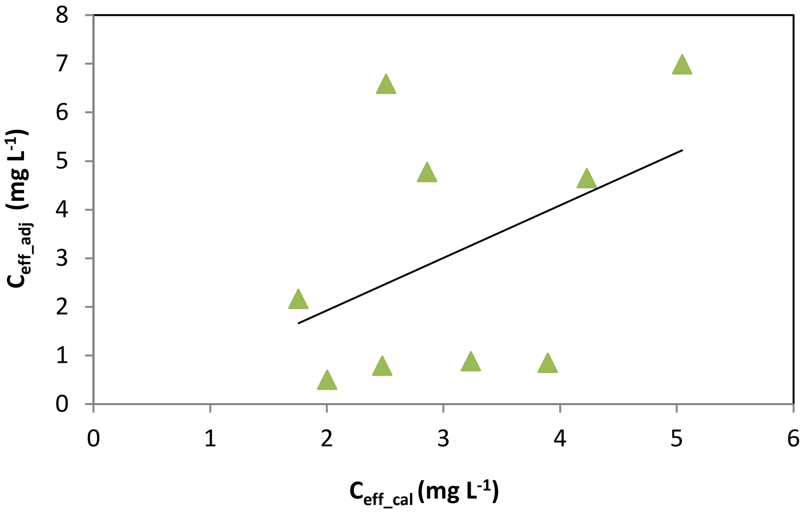

Figure B.3. Relationship between Ceff_cal and Ceff_obs for Ammonia in the SF(Summer) (*k* = 32.6 *C = 1.6 R^2=0.20)**

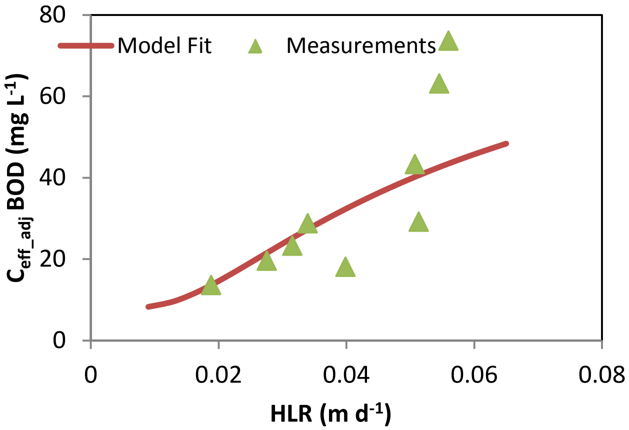

Figure B.4. Parameter Estimation for BOD_5 Removal in the SF (Summer) (*k* =19.1 *C = 8.0 *RSSE/n* = 188))**

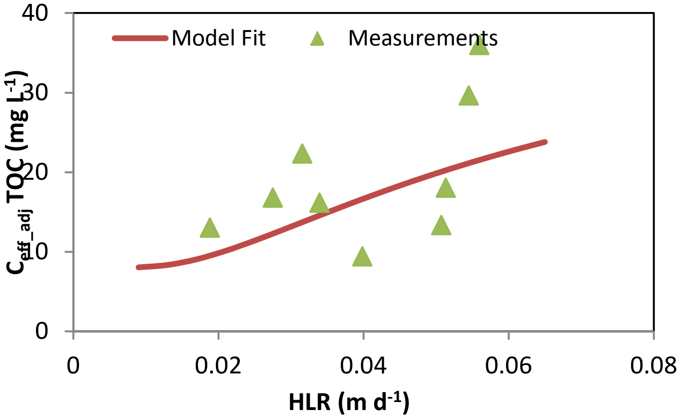

Figure B.5. Parameter Estimation for TOC Removal in the SF (Summer) (*k* = 22.8, *C = 8, *RSSE/n* = 43.4)**

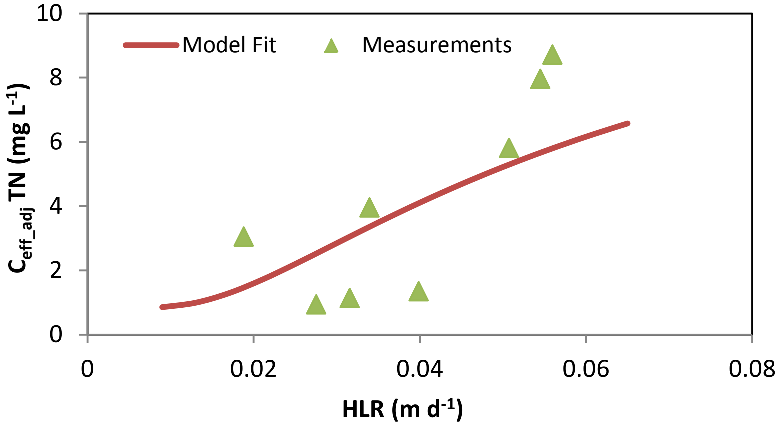

Figure B.6. Parameter Estimation for TN Removal in the SF (Summer) (*k* = 21.3, *C = 0.8* *RSSE/n* =3.3)**

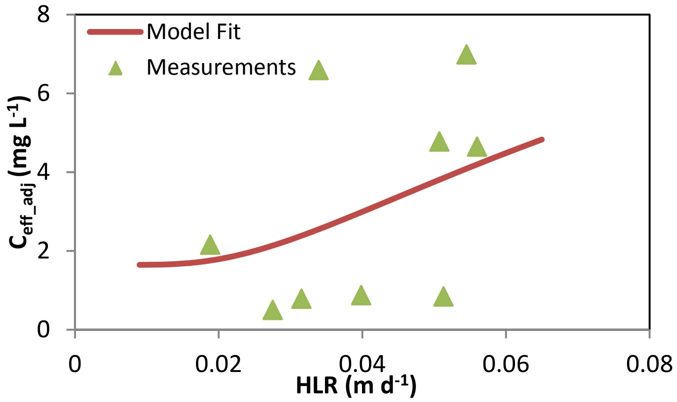

Figure B.7. Parameter Estimation for Ammonia Removal in the SF (Summer) (*k* = 32.6, *C = 1.6, *RSSE/n* =4.9)**

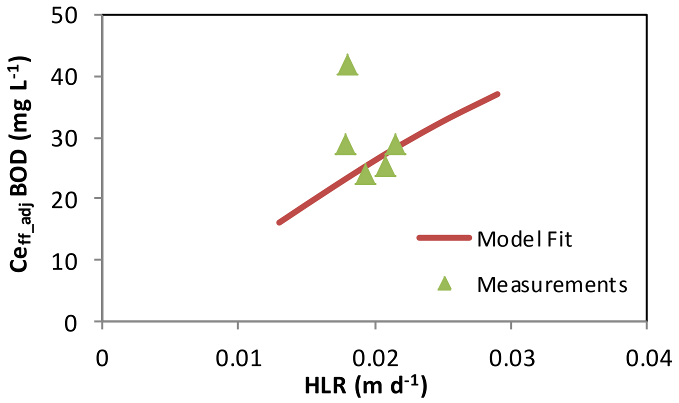

Figure B.8. Parameter Estimation for BOD Removal in the SF (Fall) (*k* = N/A)

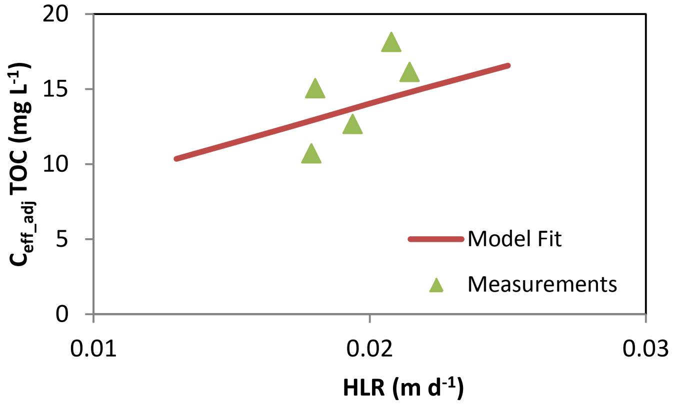

Figure B.9. Parameter Estimation for TOC Removal in the SF (Fall) (*k* = N/A)

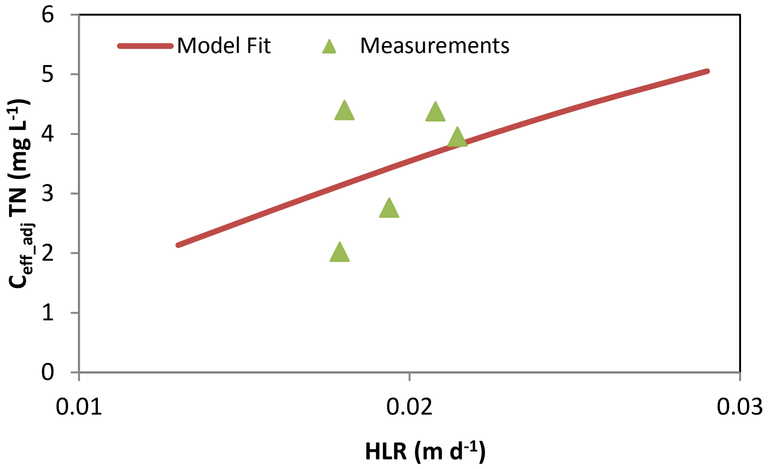

Figure B.10. Parameter Estimation for TN Removal in the SF (Fall) (*k* = N/A)