THESIS

AN ADVANCED DECENTRALIZED WASTEWATER MANAGEMENT PLANNING STUDY AND DEMONSTRATION PROJECT FOR THE CSU FOOTHILLS CAMPUS

Submitted by

Neal Thomas Gallagher

Department of Civil and Environmental Engineering

In partial fulfillment of the requirements

For the Degree of Master of Science

Colorado State University

Fort Collins, Colorado

Spring 2010

COLORADO STATE UNIVERSITY

January 26, 2010

WE HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER OUR SUPERVISION BY NEAL THOMAS GALLAGHER ENTITLED AN ADVANCED DECENTRALIZED WASTEWATER MANAGEMENT PLANNING STUDY AND DEMONSTRATION PROJECT FOR THE CSU FOOTHILLS CAMPUS BE ACCEPTED AS FULFILLING IN PART REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE.

Committee on Graduate work

____
Larry Roesner

____
Chris Goemans

____
Advisor: Sybil Sharvelle

____
Department Head: Luis Garcia

ABSTRACT OF THESIS

AN ADVANCED DECENTRALIZED WASTEWATER MANAGEMENT PLANNING STUDY AND DEMONSTRATION PROJECT FOR THE CSU FOOTHILLS CAMPUS

Expansion of development on the Colorado State University's (CSU) Foothills Campus has required examination of alternative methods to manage wastewater produced within the campus. This work builds off previous work which demonstrated that reuse of graywater and treated blackwater effluent could greatly reduce the cost of supplying wastewater treatment for the Foothills Campus (Criswell & Roesner 2005). The objective of this work was to provide insight into innovative decentralized wastewater technologies and management techniques to lay the groundwork for planning and design of optimal decentralized wastewater treatment architecture for the Colorado State University Foothills Campus. This objective was met through a planning study and a demonstration project examining anaerobic digestion of blackwater.

A planning study was performed providing four potential scenarios for management of wastewater on the Foothills Campus. Source separation was recommended for proposed development, however combined plumbing in existing development was left unaltered. Four different wastewater streams were identified by type and level of treatment necessary: blackwater, graywater, laboratory process water, and laboratory sink water. Anaerobic digestion was recommended for primary treatment of blackwater because of the renewable energy (methane biogas) and nutrient rich effluent which are produced. Constructed wetland treatment was recommended for graywater and laboratory process water, to provide a source of reusable water for irrigation or toilet flushing. Technical feasibility of treatment of graywater from a campus setting in a constructed wetland has been previously examined, showing substantial levels of treatment.

Technical feasibility of anaerobic digestion of blackwater from a campus setting is further examined in this study through a 108 L upflow anaerobic sludge blanket (UASB) treating raw blackwater from a building on the Foothills Campus. Reactor operational OLR varied between 0.21-0.39 kg COD/m^3·d and HRT varied between 2.6-4.0 days during the study period. Total reactor operational time was 108 days at an effluent temperature of 28°C. Substantial removal of COD (72%), TSS & VSS (95%), and indicator organisms (1.4 log E. coli & 1.1 log fecal coliforms) was achieved over the study period. Effluent containing 79 mg/L dissolved ammonia nitrogen showed potential for use as fertilizer. Methane biogas produced during digestion (137 L CH_4/kg COD_{input}) provided potential as a source of renewable energy. Overall performance of the UASB was sufficient for pretreatment of Foothills Campus blackwater. However, further examination of effluent, solids, and biogas reuse potential is necessary to determine supplementary treatment requirements and desired applications for extracted resources.

Neal Thomas Gallagher
Civil and Environmental Engineering
Colorado State University
Fort Collins, Colorado 80523
Spring 2010

ACKNOWLEDGEMENTS

I am extremely grateful to my advisor Dr. Sybil Sharvelle for her guidance, inspiration, and support. It has been an honor working under her supervision. I would also like to thank Dr. Larry Roesner and Dr. Christopher Goemans for their direction and counsel. In addition, I am very thankful to those who helped in and out of the laboratory, forming ideas, constructing reactors, and performing experiments, most notably Meg Hollowed, Sarah Breidt, Luke Loetscher, and Adam Jokerst. I would also like to thank Karin Streifel for her patience and friendship throughout the entire process. Most importantly I extend tremendous gratitude to my parents, Sarah and Mark Gallagher for their love, encouragement, and continual confidence in me. To my parents I dedicate this work.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ACRONYMS

ADF – Average daily flow
ADWM – Advanced decentralized wastewater management
bCOD – Biodegradable chemical oxygen demand
BOD – Biological oxygen demand
BW – Blackwater
CDC – Center for Disease Control
COD – Chemical oxygen demand
CSTR – Completely stirred tank reactor
CSU – Colorado State University
dCOD – Dissolved chemical oxygen demand
dIC – Dissolved organic carbon
DOC – Dissolved organic carbon
dTC – Dissolved total carbon
dTN – Dissolved total nitrogen
DWWTP – Decentralized wastewater treatment plant
FWS – Free water surface
GC – Gas chromatograph
GPD – Gallons per day
GPCD – Gallons per capita per day
GSF – Gross square feet
GW - Graywater
HRT – Hydraulic retention time
KR – Kitchen refuse
MBR – Membrane bio-reactor
MCDA – Multi-criteria decision analysis
OLR – Organic loading rate
S_I, S_{II}, S_{III}, S_{IV} – Chapter 2 treatment scenarios
SM1, SM2 – Sewer main 1, sewer main 2
SRT – Solids retention time
SSF – Subsurface flow
TN – Total nitrogen
TOC – Total organic carbon
TP – Total phosphorus
TR – Treatment region
TS – Total solids
TSS – Total suspended solids
UASB – Upflow anaerobic sludge blanket
VCP – Vitrified clay pipe
VFA – Volatile fatty acids
VS – Volatile solids
VSS – Volatile suspended solids
WWTP – Wastewater treatment plant

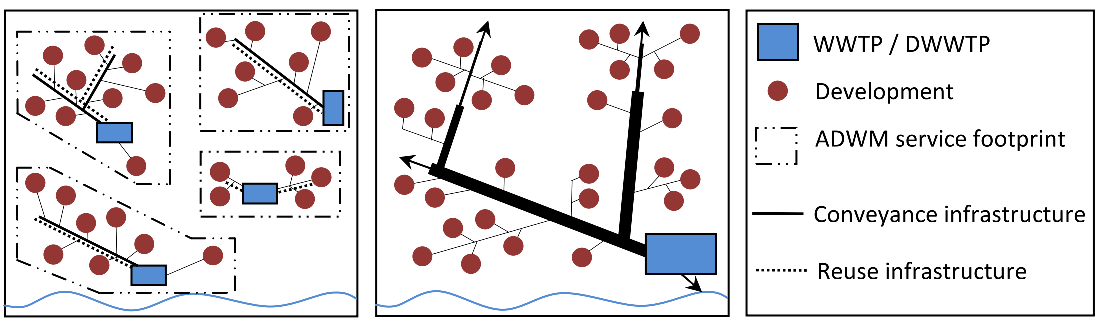
1.0 INTRODUCTION

1.1 Background

Centralized wastewater management is the most commonly encountered wastewater management strategy for large towns and cities in the developed world. Decentralized wastewater management, frequently consisting of aging and poorly maintained septic systems, is often encountered in rural areas, less densely populated regions, and municipal fringe developments where centralized wastewater collection infrastructure was unaffordable or unfeasible at the time of development (Anderson & Otis 2000). However, decentralized treatment technology and management advancements have improved performance to the point where decentralized systems can be considered as permanent solutions to wastewater management (Anderson & Otis 2000). Economies of scale make centralized treatment of wastewater cost effective per capita, yet larger service areas require burdensome collection infrastructure and far out projections into direction and quantification of future growth, giving rise to potential substantial diseconomies of scale. Advanced decentralized wastewater management (ADWM), an ideology explored in this work, incorporates innovative wastewater management and technology concepts to increase the treatment efficiency, cost effectiveness, and reliability of cluster and individual decentralized wastewater management systems. ADWM offers sustainable, effective, and reliable solutions to wastewater management in the developed and developing world.

Centralized and decentralized wastewater management concepts differ in many capacities including, but not limited to wastewater composition, treatment technologies, service area, infrastructure burden, planning considerations, and regulatory requirements. Centralized wastewater management can be defined as collection and treatment of diluted wastewater from a large population area such as one or more towns or cities, or portions of larger cities. Treated wastewater and organic solids from centralized facilities are typically assimilated by the environment through discharge into waterways and application onto land (Anderson & Otis 2000). Decentralized wastewater management serves individual, or groups of two or more homes using onsite individual or cluster treatment systems. Consideration of cluster treatment under decentralized wastewater management draws into question the 'line' between decentralized and centralized wastewater management (Pinkham et al. 2004). Cluster treatment can be differentiated by the relative size (commonly serving <400 population equivalents) or treatment region (serving dense groups of development within a community instead of substantial areas of entire communities) (Figure 1.1).

Figure 1.1: Comparison of ADWM and centralized wastewater management



(a) Cluster ADWM (b) Centralized

The existing standard of centralized transport and treatment of wastewater is often not the most sustainable solution to wastewater management (Elmitwalli et al. 2006). Further, inhibitive costs of current wastewater management strategies i.e. centralized management are a major obstacle to providing adequate sanitation worldwide (Lettinga et al. 2001) (Varis & Somlyódy 1997). New innovative management solutions are necessary to provide long term cost effective solutions in new and rehabilitated wastewater management systems. A study examining the costs and benefits of decentralized wastewater management found that the optimal architecture of wastewater management would benefit from integration of principles from centralized and decentralized concepts i.e. cluster type decentralized systems (Pinkham et al. 2004). In a 1997 report to congress, the United States Environmental Protection Agency (EPA) noted that well managed decentralized wastewater treatment systems, including onsite (septic) and small community cluster treatment systems, were an economical and long-term option to meet both public health and water quality goals (EPA 2009).

Decentralized management of wastewater facilitates utilization of wastewater as a valuable resource, instead of a pollutant, which when properly managed can provide a source of locally produced reusable water, fertilizer, and energy, in addition to improvements in public health and sanitation. Management of decentralized systems to incorporate concepts that enhance the value of wastewater provides a sustainable management solution which delivers positive economical, social, and environmental benefits to communities.

Decentralized systems benefit largely from reduced size, length, and reach of conveyance infrastructure. Reduced infrastructure burden provides incentive for source separation, a concept which has potential to increase treatment efficiency through application of specific treatment technologies which target specific pollutant streams, e.g. by separating out feces, urine and organic refuse (blackwater), the majority of pharmaceuticals found in wastewater are concentrated into a reduced volume for specific treatment. Use of anaerobic digestion as a treatment technology can produce biogas, providing a source of heat or energy. Because nutrients are not reduced in anaerobic digestion, composted solids and disinfected effluent can be used as a direct source of fertilizer. Water reuse is encouraged in decentralized systems because separated water from hand sinks, showers, and clothes washers (graywater) requires relatively minimal treatment for reuse application and is a readily available source of water for irrigation or other uses. Incorporating reuse on a household level eliminates the need to convey graywater to a treatment facility, reducing the size of treatment processes and size and reach of conveyance infrastructure.

Defined service areas of decentralized systems can reduce planning projection errors, prevent unutilized capital dollars by running at or near capacity on startup, and allow capital to be released in accordance with development. Smaller service area footprints of decentralized systems reduce environmental impacts of construction, allow wastewater treatment systems to be tailored to match wastewater composition and the needs of wastewater producers, allow new technologies to be incorporated quicker, decrease infrastructure costs, and can greatly reduce the impacts of effluent discharge on receiving waters by extraction of nutrients and organics for beneficial uses.

To overcome obstacles limiting incorporation of ADWM concepts, knowledge transfer amongst professionals on permitting, planning, and design stages is necessary. Regulations are also a major obstacle. Graywater reuse for irrigation or toilet flushing, nutrient extraction from urine, and other ADWM concepts are relatively new tools which are not currently implemented on a large scale. State and local regulations will take time to develop, improve, and become less unnecessarily protective (as most regulations initially are when new concepts are introduced) and begin to match legislation with scientifically determined human and environmental risks. Further, increased full scale implementation of innovative ADWM concepts is necessary to overcome these obstacles.

The work presented in this thesis is intended to provide insight into innovative decentralized wastewater technologies and management techniques to encourage planning and design of optimal wastewater treatment architecture for the Colorado State University (CSU) Foothills Campus. ADWM is not meant to be a confining ideology, or a newly coined acronym, rather it is solely used within this document to denote the vast contrast which modern innovative and 'advanced' decentralized wastewater treatment technologies and management concepts make to outdated and largely ineffective decentralized wastewater management concepts i.e. septic systems. These innovative and advanced concepts provide great potential to increase the sustainability, long term cost effectiveness, and reduce environmental impacts of modern wastewater management.

Concepts surrounding ADWM are presented in this thesis through a planning study which provides options for ADWM in a campus setting and a demonstration project which examines the potential for anaerobic digestion of blackwater in ADWM. This research expands upon previously performed planning work for the Foothills Campus by providing four options for incorporation of ADWM concepts for wastewaters generated on the Foothills Campus of Colorado State University (CSU) (Criswell & Roesner 2005). Anaerobic treatment of blackwater and wetland treatment of graywater is recommended for application in each option. Insight into the performance of constructed wetland campus graywater treatment is evaluated in a study under completion at CSU. Performance of a demonstration scale anaerobic digester for treatment of blackwater is examined in this work. A decision analysis model is discussed for selection of the most site appropriate process to treat blackwater and a fundamental operational cost comparison for various treatment options is provided.

1.2 Advanced Decentralized Wastewater Management

Exploration of economic, social, and environmental feasibility of wastewater systems during the planning and management steps of wastewater architecture are important considerations to choosing appropriate wastewater treatment and management architecture. Considering ADWM concepts presents an alternative to conventional centralized architecture which can help advance economic, social, and environmental feasibility of wastewater systems. ADWM considers concepts including source separation, graywater reuse, and anaerobic digestion of blackwater for production of biogas and recovery of nutrients to improve the overall performance of wastewater management.

1.2.1 ADWM Costs and Benefits

Benefits and costs of wastewater management strategies are difficult to catalogue due to the overtly complex nature of such systems. The achievable benefits and associated costs vary greatly by individual application and site location. Decentralized wastewater management offers many areas where benefits can be attained through incorporation of advanced and innovative technologies which often are not cost effective for centralized systems. These technologies intend to increase system robustness and resiliency, reduce required maintenance and operations, and increase the overall positive impacts to user communities and the environment. The majority of benefits and costs achievable by ADWM are more effectively realized by cluster systems when compared to individual systems due to economies of scale. The following bulleted list identifies many of the potential benefits which can be achieved by properly designed, planned, constructed, and operated decentralized cluster wastewater treatment systems incorporating ADWM concepts (Pinkham et al. 2004):

- **Defined service population available during planning and design:**
  - Diminished impacts of planned capacity errors
  - Shorter population projections
  - Reduced likelihood of idle infrastructure capacity
  - Necessary capital dollars known upfront
- **Capacity built as needed:**
  - Capital is invested incrementally as development occurs
  - Minimal trapped equity in unused capacity
  - Initial stakeholder debt reduced
  - Rapid response to technological change
  - Capacity is not overbuilt to serve future development
  - Infrastructure flows closer to capacity at startup, reducing infiltration
  - Sewers do not dictate direction of development, sprawl can be prevented

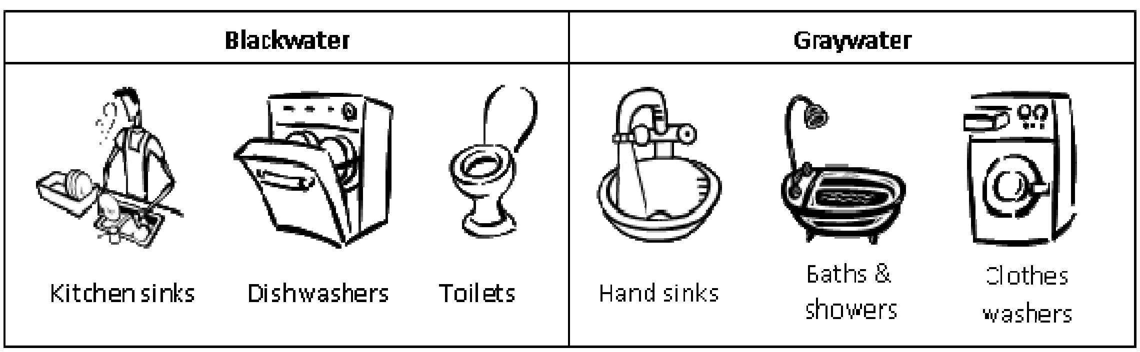
- **Smaller service areas:**
  - Reduced environmental impacts
  - Less wasted infrastructure spans across undevelopable space
  - Reduced force mains and lift stations
  - Treatment tailored to match wastewater quality and end use
  - Reduced infrastructure size and length per capita
- **Cluster systems serve groups of similar producers (e.g. communities):**
  - Wastewater streams more homogeneous, predictable
  - Infrastructure and treatment processes tailored to meet local needs
  - Level of treatment is reduced to meet requirements for desired end product use
  - Individuals have better understanding of environmental impacts associated with wastewater production and treatment byproducts
- **Process failure has reduced impacts:**
  - Smaller distributed systems have reduced consequences of failure
  - Extraction and reuse of byproducts (e.g. nutrients) reduce negative environmental impacts on receiving waters
  - Less vulnerable to intentional sabotage and natural hazards
- **Source separation is encouraged:**
  - Source separated wastewater is more homogeneous
  - Specific treatment processes can target specific wastewater streams
  - Conveyance infrastructure and treatment process size reduced by household graywater separation
  - Innovative collection infrastructure technologies (e.g. vacuum) may make sense for concentrated streams traveling shorter distances for treatment
  - Separate sources can be treated only to levels required for desired end use
  - Reduced treatment requirements can reduce energy inputs
  - Allows local extraction and application of valuable resources from locally produced wastewater, e.g. biogas, nutrient rich effluent
- **End use of treatment products is more dynamic:**
  - Nutrients can be extracted and reused for fertilizer
  - Biogas can be used to supplement reactor heat and community needs
  - Organic solids can be sold for compost or used locally
  - Extraction of valuable end products is more efficient in source separated and concentrated streams

1.2.2 Wastewater Source Separation

Wastewater source separation is the segregation of wastewater sources by relative quality and risk to human health. Source separation is a concept which promotes improvement the overall performance of ADWM system architecture. Sources of household wastewater are typically broken into specific wastewater streams by characteristics including organic loading, level and type of treatment required for end use, and potential for resource extraction. Household wastewater sources are typically separated into two factions, graywater and blackwater (Figure 1.2). Blackwater can be further broken down into yellowwater and brownwater. Rainwater is also an available water resource which can be incorporated into ADWM architecture:

- Graywater: shower, hand sink, and clothes washer wastewaters
- Blackwater: feces, urine, flushwater, dishwasher, and organic kitchen refuse
  - Yellowwater: urine and flushwater
  - Brownwater: feces, flushwater toilet paper, dishwasher, and organic kitchen refuse
- Rainwater: rainfall runoff from building roofs or other surfaces

Figure 1.2: Typical blackwater and graywater sources



Household wastewater contains many valuable constituents essential to life including water, nitrogen, phosphorus, potassium, sulfur, magnesium, and other trace elements which are difficult to recover in the diluted flows of traditional wastewater management (Otterpohl et al. 2003). Wastewater source separation increases the overall performance of wastewater management by preventing entropy gain through dilution of wastewater (USEPA 2007) and isolating waste streams with higher treatment requirements and higher concentrations of valuable nutrients and organics (blackwater) from wastewater that requires relatively low treatment levels and which contributes a larger volume of annual flows (graywater). Treatment of undiluted separated wastewater streams is more resource efficient than treatment of diluted wastewater (Larsen & Gujer 1996). In addition, water reuse is encouraged by graywater separation since relatively minimal treatment is typically necessary prior to application. In this section, graywater and blackwater are defined in general terms as commonly observed in practice. Sources are redefined for specific adaptation to Foothills Campus wastewater characteristics in Chapter 2.

1.2.2.1 Graywater

Graywater can be generally defined as wastewater generated in hand wash sinks, clothes washers, and showers. Kitchen sinks and dishwashers do not contribute to graywater due to potential to contain higher levels of pathogens and organics, mainly from food particles. Introduction of organics into the graywater stream increases the biological activity and potential for oxygen depletion in graywater harvesting systems (Roesner et al. 2006).

Graywater accounts for approximately 30-50% of total generated household wastewater (O'Connor et al. 2008) (Roesner et al. 2006). Graywater contribution as a fraction of total household wastewater can be broken down as, baths (1.7%, 4.5 L/cap/day), faucets (15.7%, 41 L/cap/day), showers (16.7%, 44 L/cap/day), and clothes washers (21.6%, 57 L/cap/day) (Roesner et al. 2006).

Separation of graywater from an existing treatment system can greatly reduce conveyance and treatment system loading. Substantially less treatment is required for graywater relative to blackwater. Graywater treatment requirements vary in accordance with desired end use and local regulations. Minimum treatment is the use of coarse filtration, e.g. a mesh screen, for removal of large matter such as hair, thread, and lint (Roesner et al. 2006). For use in toilet flushing, graywater should undergo filtration and disinfection to attain acceptable fecal coliform removal (Al-Jayyousi 2003). Graywater used for irrigation can vary in treatment depending on the potential for human contact and what is being irrigated. Filtration, constructed wetlands, and biological trickling filters are viable options for treatment of graywater with disinfection a recommendation if direct human contact with treated effluent is possible.

1.2.2.2 Blackwater

Blackwater is wastewater from sources high in organics, pathogens, and emerging contaminants including toilets, kitchen sinks, garbage disposals, and dishwashers. High concentrations of organics and nutrients found in blackwater facilitate extraction of valuable products from concentrated blackwater including methane biogas, nitrogen and phosphorus. Blackwater contains a majority of the pathogens and emerging contaminants found in wastewater e.g. pharmaceuticals and other endocrine disrupting compounds. Separating these contaminants which require relatively high and specific treatment at the source greatly reduces the volume of wastewater which is of greater health and environmental risk. Pathogens and emerging contaminants are confined to a treatment process tailored for their destruction.

Approximately 97% of nitrogen, 80-90% of phosphorus, 77% of total solids, 66% of potassium and 44-59% of total organic load of wastewater can be contributed to blackwater, which requires approximately 31% of household freshwater (Lopez Zavala et al. 2002) (Otterpohl et al. 2003). High organic loading and pathogen levels in blackwater require high levels of treatment to meet quality regulations and protect public health and the environment. Decentralized technologies to treat blackwater exist incorporating aerobic and anaerobic biological processes. Anaerobic onsite and cluster treatment concepts include high rate anaerobic digesters e.g. upflow anaerobic sludge blanket (UASB), UASB septic tanks (Luostarinen & Rintala 2007), dehydrating eco-toilets (Winblad et al. 2004), and composting toilets (Werner et al. 2009). Anaerobic digestion of blackwater does not substantially reduce nutrients, which is beneficial for effluent reuse in irrigation. Anaerobic digestion also produces methane biogas which can be used locally as a source of heat or energy. Blackwater can be further separated into yellowwater and brownwater.

1.2.2.3 Yellowwater

Yellowwater is urine separated from feces. Urine separating toilets or urinals can be used to collect yellowwater. Urine separation isolates a small volume of flow (1.4 L/person·d (Larsen et al. 2009)) containing a large percentage of nutrients, approximately 87% of nitrogen, 50% of phosphorus, and 54% of potassium found in wastewater (Otterpohl et al. 2003). High nutrient loads and low treatment requirements make application of yellowwater an inexpensive source of fertilizer in agriculture including applications in crop irrigation and aquaculture (SUSANA 2009). Urine by itself is generally sterile, however some infections exist that can cause passage of pathogens into urine (Eriksson et al. 2002). For this reason, if urine is to be applied directly as a source of nutrients to irrigated crops, a storage time of 1-6 months is recommended dependent on storage temperature and type of crop irrigated (Winblad et al. 2004). Urine source separation can offer an alternative to expensive nutrient removal in treatment plants. At 60% catchment of urine within a municipal system, the C:N:P (carbon:nitrogen:phosphorus) ratio is near optimal for biological growth (Larsen et al. 2009).

1.2.2.4 Brownwater

Brownwater consists of feces, toilet paper, and flushwater from toilets which incorporate urine separation, and can be expanded to include organic kitchen refuse. Feces and organic kitchen refuse contribute a majority of the organic load and pathogens found in blackwater. Nearly optimal C:N:P ratios for biological microorganism growth can be achieved in brownwater through separation from yellowwater (Larsen et al. 2009). Treatment requirements and processes for brownwater are comparable to blackwater. Anaerobic treatment of brownwater may be enhanced through separation of yellowwater since the pH raising effect of conversion of organic nitrogen (found in yellowwater) to ammonia is greatly reduced.

1.2.2.5 Rainwater

Rainwater is also a harvestable water resource where permissible by law. Rainwater can be collected from household rooftops and stored in tanks or cisterns. Captured rainwater can be applied for use in landscape or garden irrigation and for toilet flushing.

1.2.2.6 Water Reuse

Water reuse applications cover a wide array of options including agricultural and landscape irrigation, cooling water for industrial processes and power generation, groundwater recharge, snowmaking, fire protection, and toilet flushing. Treatment required for reusable water is governed by the end use of the water to ensure protection of public health and the environment (Tchobanoglous et al. 2003). Agricultural irrigation is the largest consumer of reusable water at 2.72 Mm^3/d, followed by industrial and thermoelectric reuse (Tchobanoglous et al. 2003). Quality of treated effluent varies dependent on many factors including influent wastewater characteristics, treatment process design, and operational efficiency. Varying effluent qualities from specific treatment processes can be combined or used for specific purposes depending on desirable end use.

Graywater can be used directly without treatment for applications such as landscape irrigation. Treatment of graywater improves quality and increases options for reuse, such as toilet flushing. At this time, reuse of graywater is most likely the most widely incorporated source separation concept in the United States on a household scale. It can be inferred that this is due to the low levels of treatment necessary and the ease of household plumbing alteration to collect graywater. Reuse of graywater for irrigation can reduce potable demands by 30-50% (O'Connor et al. 2008) (Roesner et al. 2006). Residential and commercial graywater reuse is a growing practice gaining increasing popularity and acceptance in the United States and on an even larger scale internationally (Roesner et al. 2006). States including Arizona, California, New Mexico, Utah, and Texas have legalized graywater reuse for landscape irrigation (Roesner et al. 2006). Hotels, university dormitories, and businesses are using graywater for toilet flushing, residences are being plumbed to divert graywater to holding tanks and treatment systems for landscape irrigation and entire communities are being designed around separate collection, treatment and reuse of graywater (Hochedlinger et al. 2008) (March et al. 2004) (Roesner et al. 2006).

Blackwater requires greater intensity of treatment before treated effluent can be reused for beneficial purposes. Treated effluent from anaerobic digesters treating blackwater may contain elevated levels of volatile fatty acids which at high levels could cause harm or death of irrigated vegetation. Nitrogen is not removed in anaerobic digesters and minimal phosphorus removal is experienced. For this reason, digester effluent is high in nutrients and after disinfection is a viable source of nutrient enhanced irrigation water.

1.2.2.7 Energy Production

Source separation isolates the majority of organic carbon in wastewater into the blackwater stream. Solids and organics concentrations in blackwater are typically feasible for anaerobic digestion, an energy producing wastewater treatment process.

1.3 History of Wastewater Management

Prior to construction of public water works, availability of water in households was limited, resulting in far smaller wastewater volume produced per capita. Wastewater flows consisting mainly of fecal matter, urine, and minimal washwater allowed primitive decentralized privy-vault and cesspool systems to effectively manage the low per capita quantity of wastewater produced both in urban and rural areas. In the mid nineteenth century, several factors contributed to the need for new wastewater management methods. The switch from decentralized privy-vault and cesspool wastewater management in the later nineteenth century to modern centralized systems was influenced heavily by (Burian et al. 2000):

- Overtaxing of existing systems due to increasing population densities
- Public health concerns (e.g. disease outbreaks from sewage contamination of water and air)
- Construction of public water works greatly increased quantity of wastewater produced
- Perceived lower costs over the lifetime of the centralized system
- General lack of alternative options

The first modern centralized wastewater management system was constructed in Hamburg, Germany in 1843 and Chicago and Brooklyn followed shortly, constructing centralized wastewater infrastructure in the 1850's (Burian et al. 2000). Initially, most centralized sewerage systems received wastewater from households, businesses and other producers of wastewater and emptied into receiving waters without treatment. This alleviated many public health problems upon initiation. However, as wastewater production increased, the detrimental effects of the pollutants on receiving waters also increased. The need for treatment of wastewater was recognized on a large scale in the early twentieth century due to encouragement from public health groups, businesses, media, and regulation (Burian et al. 2000). Because centralized infrastructure was already in place, the obvious solution was to implement end of pipe treatment.

In the United States, post WWII, suburban areas began to increase in population and centralized infrastructure was either extended to serve these areas or septic systems were installed where expansion of existing or construction of new centralized systems was not feasible. Water Pollution Control Acts of 1948 and 1972 further encouraged centralized wastewater management by providing planning, technical services, research, financial assistance, and enforcement of regulations.

In 1972, a massive $24.6 billion was authorized by the 1972 Water Pollution Control Act to finance municipal wastewater treatment systems in an effort to stem widening pollution problems across the U.S. At this period, most efforts in planning, research, and engineering had been focused on centralized wastewater management. Because little advances had been made in decentralized management and decentralized treatment technologies, this funding took focus from providing the most cost effective alternative and encouraged expansion of centralized wastewater management concepts (Burian et al. 2000).

In the 1980s reduction in federal funding for centralized wastewater systems and development and innovation in technologies for smaller scale wastewater treatment lead to reconsideration of decentralized wastewater treatment as a viable management tool. In addition, with dissipation of large federal funding for centralized solutions, greater emphasis has been placed on determination of whole-system life cycle analysis of wastewater infrastructure during initial planning stages to provide the most cost effective and sustainable alternative for wastewater management in communities and industrial applications. Research, development, and implementation of concepts including wastewater source separation, low flow and vacuum collection, water reuse, nutrient extraction, and use of anaerobic digestion for extraction of biogas have paved the way for more environmentally sustainable, reliable, and cost effective decentralized wastewater management solutions. ADWM incorporates these concepts to provide beneficial wastewater management solutions which can be highly competitive with centralized solutions and have potential to offer additional benefits to users.

1.4 Background

Although centralized wastewater management has greatly contributed to improvements in public health and the environment over the course of the past 150 years, there are inevitably concepts available which can increase the sustainability of wastewater architecture over the long term. There is a call to evaluate new concepts for water and wastewater management (USEPA 2007). ADWM is an alternative to centralized wastewater management which can improve environmental sustainability and cost effectiveness of wastewater management in the developed and developing world.

ADWM encompasses onsite systems which collect and treat wastewater from individual homes and buildings as well as cluster systems which collect and treat wastewater from multiple homes and buildings. Because development often occurs in relatively dense clusters, ADWM concepts can drastically reduce both the diameter and length of required conveyance infrastructure as compared to centralized wastewater management when incorporated in cluster treatment architecture. ADWM incorporates innovative technology and management concepts to change the way wastewater is managed and treated in decentralized systems. ADWM concepts including source separation, nutrient and biogas extraction, and treated effluent reuse can result in major improvements to the environmental sustainability and cost effectiveness of traditional centralized and decentralized wastewater management.

1.4.1 Review of ADWM Concepts

There are many concepts emerging which promote sustainable decentralized management of wastewater in line with the concept of ADWM. The following summarize the more prevalent concepts discussed in available literature.

1.4.1.1 Integrated Wastewater Planning

Integrated wastewater planning improves upon conventional wastewater facility planning by representing whole system costs and benefits of wastewater management systems (Pinkham et al. 2004). Integrated wastewater planning is a more comprehensive method of planning which gives decentralized systems adequate consideration as alternatives to centralized systems. Integrated wastewater planning is an attempt to broaden the scope of planning activities for wastewater management systems by consideration of the true implications wastewater management planning decisions.

1.4.1.2 Ecological Sanitation, EcoSAN

EcoSAN promotes extraction the water, energy and nutrient resources found in wastewater for beneficial reuse locally in agriculture and to increase sustainability of wastewater management. EcoSAN does not promote specific wastewater treatment technologies. Rather, an interdisciplinary approach is taken to provide a wastewater management solution that addresses issues including local reuse of end products, cultural acceptance and appropriateness, and community planning, contributing largely to integrated management of natural resources (Werner et al. 2009). Technologies such as urine-diversion dehydration (UDD) toilets, composting, rainwater harvesting, constructed wetlands, vacuum sewers, anaerobic digestion are often incorporated into EcoSAN projects (GTZ 2009). Source separation and preventing dilution of flow streams are two principles which optimize cost efficiency, treatment quality, and promote nutrient, energy, and water extraction and reuse in EcoSAN projects (Werner et al. 2009).

1.4.1.3 Decentralized sanitation and reuse, DESAR

DESAR is an acronym developed by Wageningen University in Wageningen, The Netherlands which focuses on the treatment of wastewater sources separately to efficiently extract and reuse nutrients, energy, and water contained within (Wageningen University 2009). Low flush vacuum toilets are used to collect and capture very high solids blackwater. Blackwater is co-digested with organic kitchen refuse anaerobically to produce biogas and nutrient rich solids which can be reused in agriculture as fertilizer.

1.4.1.4 *Decentralized wastewater treatment systems, DEWATS*

DEWATS is a concept promoted by the Bremen Overseas Research and Development Association (BORDA) which promotes finding the most case appropriate solution to providing sustainable sanitation to developing countries for flows less than 1000 m^3/d. The principles of DEWATS include low maintenance, state of the art technologies constructed with materials found locally that are designed to meet local treatment standards. Four basic technical treatment elements are used for the fundamental basis of DEWATS systems (BORDA 2009):

- Primary treatment using sedimentation and flotation
- Secondary treatment in anaerobic fixed bed reactors
- Tertiary aerobic treatment in sub-surface flow filters
- Tertiary aerobic treatment in polishing ponds

DEWATS principles set groundwork to developing sanitation technologies to provide sustainable sanitation in areas where previously, the high costs and energy requirements of conventional treatment or low performance and high maintenance requirements of low technology treatments was an obstacle.

1.4.2 Considerations to Application of ADWM Concepts

Major considerations to larger scale implementation of ADWM concepts were addressed in a 2007 report from WERF (Etnier et al. 2007):

- Lack of financial reward for using decentralized systems for designers and planners
- Designers and planners lack of knowledge of decentralized systems
- Designers and planners unfavorable perceptions of decentralized systems
- Unfavorability of the regulatory system towards decentralized systems
- Integrated systems thinking, e.g. water reuse, not generally applied to wastewater system planning
- Increased monitoring requirements

These issues well summarize the existing barriers to wide scale incorporation of ADWM concepts in full scale implementation. Legislation needs to reflect scientifically determined human risk and expand to reduce the burden of permitting innovative systems, e.g.

performance based legislation. In addition, knowledge transfer amongst professionals on permitting, planning, and design stages is necessary to reduce design delays encountered. To solve these issues, increased demonstration of the benefits of integrated and innovative ADWM is necessary to improve and expand existing regulations.

1.4.3 Example Projects

1.4.3.1 Lübeck-Flintenbreite, Germany

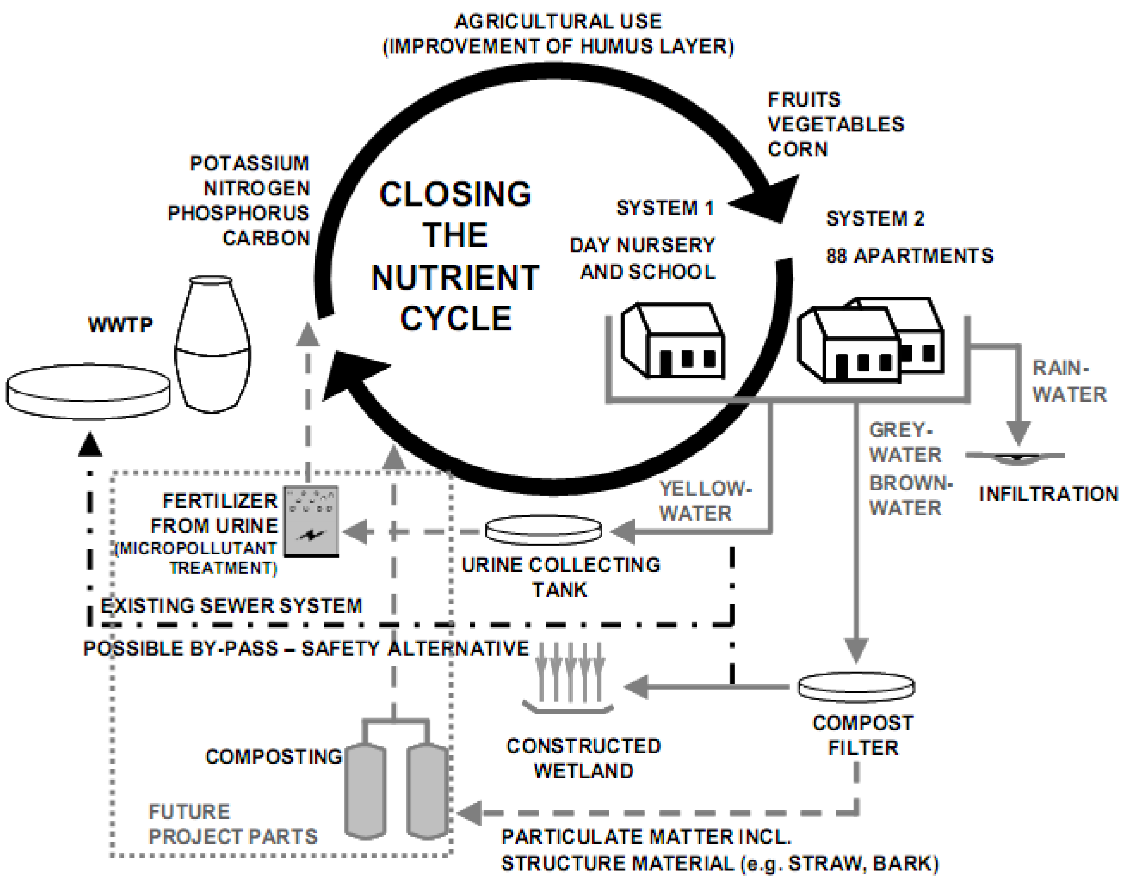
The Flintenbreite housing development in Lübeck, Germany incorporated sustainable wastewater management into design during initial planning stages. In Flintenbreite, blackwater is diverted to a central holding tank using water reducing vacuum toilets and organic kitchen refuse is combined with blackwater for digestion. Methane biogas produced in the digester is used to supplement natural gas for combined heat and power generation. Sludge from anaerobic digestion is used for agricultural application.

1.4.3.2 solarCity (Linz, Austria)

solarCity is an urban development project in the city of Linz, Austria constructed around three pillars of sustainability: economic growth, ecological balance, and social progress. The city of Linz master plan was amended in 1992 to incorporate an "Ecological Residential Area" on city owned property zoned as grassland for agricultural use (Linz 2009) (Hochedlinger, Steinmüller et al. 2008). Planning of solarCity as a sustainable development was shaped between 1995 and 1998 through architectural design competitions that promoted sustainable design concepts in landscape, energy, and infrastructure. Plans to develop 1,300 apartments over a land area of approximately 150 acres using low energy building standards were slated (Hochedlinger, Steinmüller et al. 2008). Final design incorporates housing, open city centers and courtyards, an elementary school center, and a kindergarten, aspects which also benefit residents outside of solarCity. Emphasis was placed on restoration and construction of natural and open spaces in and around solarCity.

Several advanced concepts have been applied for water and wastewater management. A pilot study involving concepts of ecological sanitation (ECOSAN) has been incorporated into solarCity Linz (Figure 1.3).

Figure 1.3: Wastewater management concepts for solarCity Linz (Hochedlinger et al. 2008)



Yellowwater, blackwater, graywater, and rainwater are managed to attempt to close the nutrient cycles. To separate yellowwater from blackwater, urine separating toilets were installed in 88 housing units and the elementary school. Future expansions of the project include micropollutant extraction from yellowwater (urine & flushwater) and production of fertilizer from wastewater. The future plan for blackwater is to send flows through a compost filter where particulate matter will be sent for composting to produce beneficial fertilizers and soils supplements.

Graywater from showers, dishwashers, and washing machines is routed through a constructed wetland for clarification and discharged into a local stream. Rainwater is routed through a system of vegetated swales, infiltration ditches, and retention basins to promote onsite infiltration. Management of rainwater was modeled considering the following priorities (Linz 2009):

- Rainwater is dealt with as and where it occurs, in a surface-oriented, decentralized system that makes the natural rainwater cycle visible and comprehensible.
- The drainage, collection and disposal of rainwater is achieved mainly by means of gutters, retention hollows and vegetated swales.
- These are integrated into a coherent, interconnected system that uses the Aumühlbach stream as a receiving water body in the southern part of the district and the alluvial meadows as a recipient in the northern part.
- The aforementioned elements of rainwater management are an integral part of the planning of the open spaces.

1.4.3.3 DWM Projects in the U.S.

Although much research has been performed examining the effectiveness and benefits of ADWM concepts (Pinkham et al. 2004), and many organizations exist which promote ADWM concepts such as source separation and water reuse (e.g. Water Reuse Foundation, The Onsite Consortium, National Onsite Wastewater Recycling Association), there are few full scale implementations of ADWM concepts in the U.S. Some notable projects ongoing within the U.S include reuse of graywater for irrigation and toilet flushing in states such as California, Arizona, New Mexico, and North Carolina, as well as a constructed wetland at CSU treating graywater produced in a campus setting.

Much of the recognition for sustainable building in the United States is achieved through design and construction of buildings in accordance with the U.S. Green Building

Council's Leadership in Energy and Environmental Design (LEED) program. Ranking of the sustainability of wastewater design in the LEED program is only a small portion of the overall points available through other technologies such as building energy efficiency and construction material recycling. For this reason, incentive may not exist for developers to incorporate ADWM concepts to the full extent in the U.S.

The Helena Building, New York, NY

The Helena Building is a LEED Gold certified high-rise apartment building incorporating a wastewater treatment and recycling system which reclaims near 163 cubic meters of wastewater per day (Gonchar 2007). Blackwater is treated in a membrane bioreactor and treated effluent is used in combination with stormwater for flushing of toilets, irrigation of rooftop gardens totaling 1,115 square meters, as well as in the cooling tower (Gonchar 2007). Several other buildings in the area have similar wastewater recycling systems including the Tribeca Green and Solaire buildings also located in Battery Park.

Port of Portland, Portland, OR

The Port of Portland is constructing a building designed to achieve a LEED Gold certification which incorporates recycling of treated effluent from blackwater for use in toilet flushing. The treatment mechanism used is the Tidal Wetlands Living Machine designed by Worrell Water Technologies (Worrell Water Inc. 2009), capable of producing tertiary treated water. Blackwater and graywater are treated biologically in a system similar to a subsurface flow constructed wetland.

1.5 Research Objectives

The overarching objective of this project was to provide insight into innovative decentralized wastewater technologies and management techniques to lay the groundwork for planning and design of optimal decentralized wastewater treatment architecture for the Foothills Campus. This project consists of two distinct elements:

- Planning (Chapter 2)
- Demonstration (Chapters 3 & 4)

Each element is presented as a separate entity with specific objectives. The planning portion of this study is presented in Chapter 2 and provides insight into wastewater management during expansion of the Foothills Campus. The demonstration portion of this study spans Chapters 3 & 4 and examines anaerobic treatment of blackwater.

1.5.1 Sustainable Wastewater Management Planning Study

The objective of the planning portion of this study was to present alternatives for infrastructure configuration and treatment process design which incorporate innovative and sustainable concepts to capture and treat wastewater produced within the Foothills Campus during expansion. A preliminary plan for decentralized wastewater treatment on the Foothills Campus is presented and treatment technologies and concepts are offered which will increase the overall sustainability of wastewater management. Appropriate location and sizing of treatment processes is outlined. The City of Fort Collins has placed a discharge limit on Foothills Campus wastewater volume. The Foothills Campus is nearing this limit. Once the limit is tripped, the City of Fort Collins will require CSU to make costly improvements to their wastewater infrastructure most likely to include a new large interceptor and updates to their treatment facility. This planning study is intended to provide a cost effective alternative solution to capture and treat wastewater onsite using innovative and sustainable concepts.

1.5.2 Anaerobic Digestion of Blackwater Study

The objective of the demonstration portion of this study was to examine potential for anaerobic digestion to serve as an effective and sustainable blackwater treatment technology for treatment of Foothills Campus blackwater. Feasibility of anaerobic digestion as a blackwater treatment technology and beneficial outputs including methane production and effluent nitrogen concentrations were examined. Minimal research has been performed on blackwater from conventional toilets and no information was available on anaerobic treatment of blackwater from a research campus setting. This project was intended to contribute to the growing knowledge base of anaerobic digestion of blackwater.

2.0 FOOTHILLS CAMPUS SUSTAINABLE WASTEWATER MANAGEMENT PLANNING STUDY

The objective of this study was to present Colorado State University (CSU) with alternatives for infrastructure configuration and treatment process design which incorporate innovative and sustainable concepts to capture and treat wastewater produced within the Foothills Campus during expansion. ADWM concepts, which intend to improve the overall performance of wastewater management including wastewater source separation, treated effluent reuse, nutrient recycling, and extraction of renewable energy in the form of methane biogas makeup the foundation of the plan. Four scenarios have been outlined which manage conveyance and treatment of Foothills Campus wastewater in varying capacities onsite.

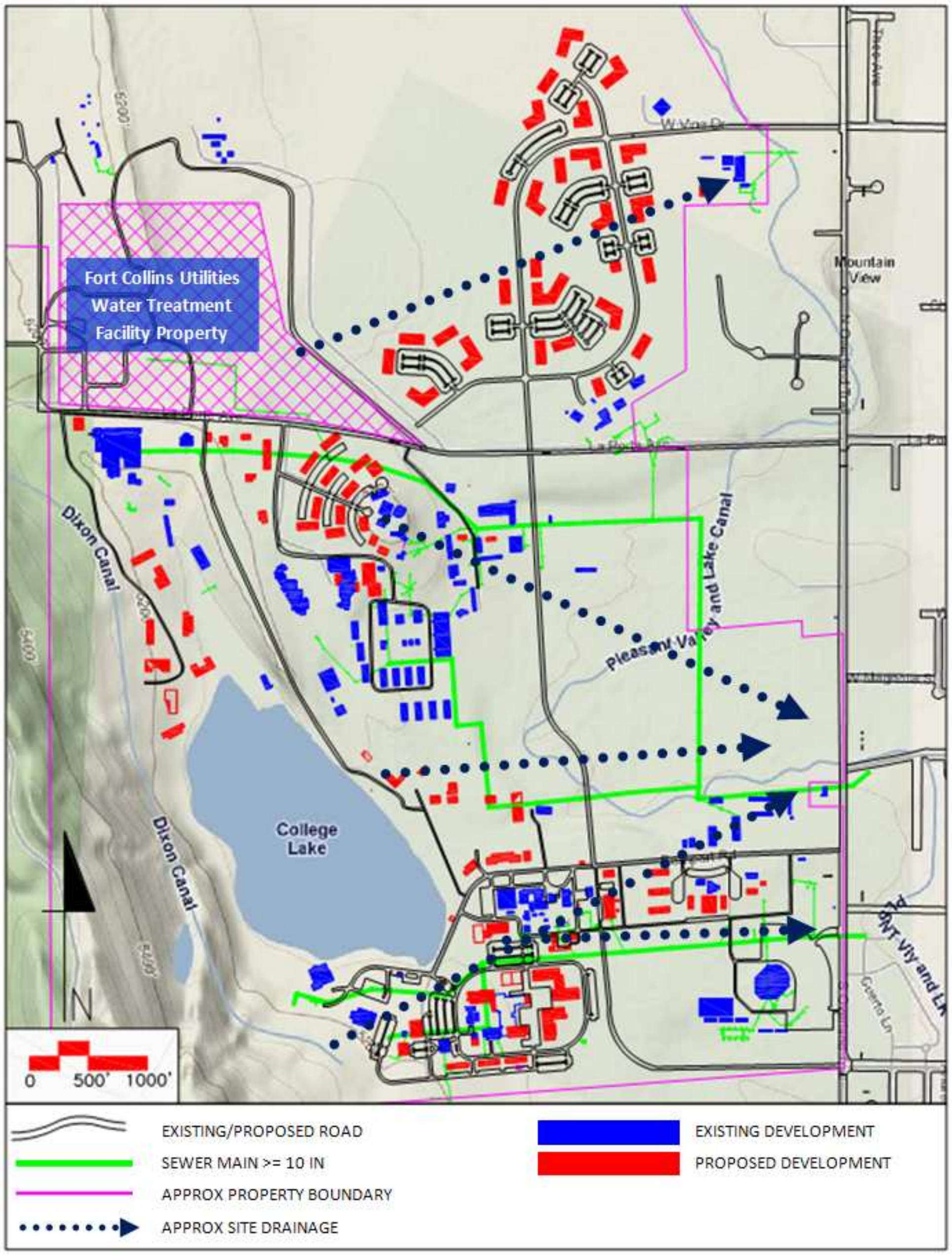
2.1 Background

2.1.1 Foothills Campus Study Area

The Colorado State University (CSU) Foothills Campus consists of more than 70 buildings containing research facilities for both CSU and other governmental agencies. CSU intends to expand the Foothills Campus with potential to increase total building square footage from 1.0 MSF to 3.5 MSF at complete buildout. An overview of the Foothills Campus with existing development in blue and proposed development in red shows the general drainage direction of the site (Figure 2.1).

With the expansion of the Foothills Campus, CSU aims to demonstrate environmental sensitivity and sustainability (RNL Design 2003). By minimizing or reducing export of resources contained in Foothills Campus wastewater, CSU can effectively reduce environmental impacts of Foothills Campus expansion. Better management of water, wastewater, and byproducts of wastewater promotes sustainability. Treated effluent can be reused for non-potable applications such as irrigation or toilet flushing. Biogas can be extracted from biosolids and used to produce green energy or steam for heat.

Figure 2.1: Foothills Campus existing site layout



Through management of Foothills Campus wastewater onsite, CSU has the opportunity to illustrate, promote, and advance sustainable water and wastewater management practices for new development throughout the entire arid West. This planning study presents an overview of four alternative wastewater management scenarios for expansion of the Foothills Campus, which intend to further environmental sustainability at CSU.

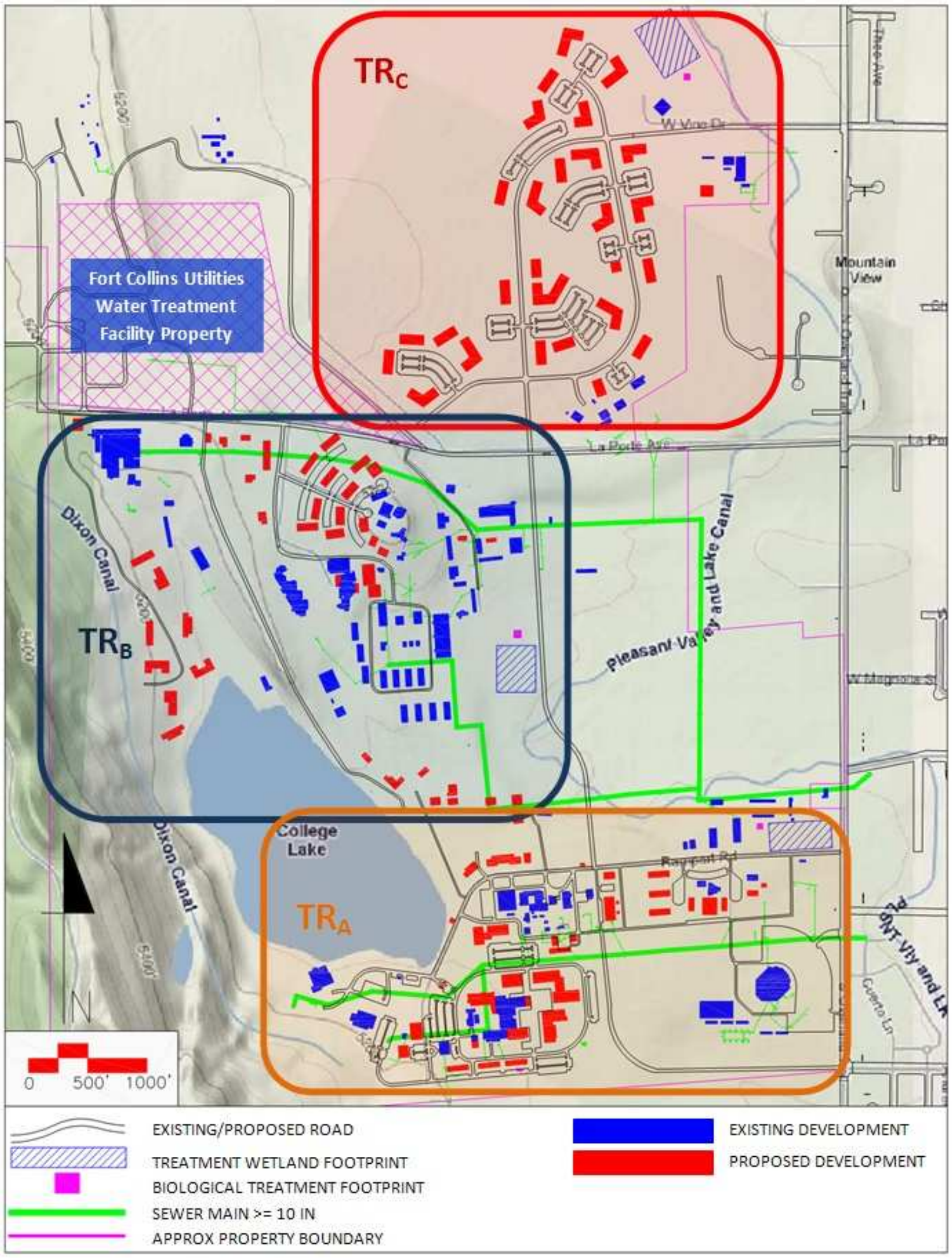
2.1.2 Development Scenarios & Treatment Regions

Decentralized wastewater management on the Foothills Campus could be designed to incorporate one main onsite treatment facility, or three smaller onsite facilities. If three facilities are chosen, the layout of existing development, wastewater infrastructure, and general topography on the Foothills Campus presents three identifiable regions which could each be served by a single facility. Three smaller onsite facilities may reduce the length of conveyance infrastructure required from building to facility and facility to reuse application. Areas potentially served by these three separate treatment facilities are identified in this report as treatment regions, or TR. The three treatment regions are referred to as TR_A, TR_B, and TR_C (Figure 2.2).

Four wastewater management scenarios have been identified, presenting varying onsite management options for Foothills Campus wastewater. Scenarios are referred to as Scenario I - Scenario IV (S_I-S_{IV}). In S_I, S_{II}, and S_{III}, the Foothills Campus is divided into three treatment regions. Wastewater from each treatment region is routed to the treatment region's own wastewater treatment facility. Wastewater from buildings within a treatment region is captured and treated within that region, unless otherwise noted in the scenario description. A figure is provided in each scenario showing treatment region boundaries. Geographical boundaries of treatment regions are the same within each scenario. However, specific routing of wastewater from existing and proposed development changes in each scenario.

TR_A represents the southernmost cluster of development within the Foothills Campus. This treatment region serves development generally adjacent to Rampart Road. TR_A contains existing research centers including the Center for Disease Control (CDC), Animal Reproduction Biotechnology Laboratory (ARBL), and the B.W Pickett Equine Center. TR_B represents the cluster of development centrally located within the Foothills Campus.

Figure 2.2: Treatment regions on the Foothills Campus



Existing development within TR_B includes the Engineering Research Center (ERC), National Wildlife Research Center (NWRC), Atmospheric Science complex (ASC) and the

Colorado State Forest Service (CSFS). Sewer main SM_2 services existing buildings within TR_B. TR_C represents the northernmost cluster of development within the Foothills Campus and encompasses development north of Laporte Avenue. Minimal existing development is located within TR_C.

2.1.3 Existing Foothills Campus Wastewater Infrastructure

Treatment of Foothills Campus wastewater is currently provided by the City of Fort Collins Utilities. Foothills Campus connects to the Fort Collins sewer system via a Fort Collins owned sewer main running parallel to Overland Trail. The majority of wastewater from the Foothills Campus existing buildings is routed to two sewer mains. The first sewer main (SM1) is a 12" vitrified clay pipe (VCP), which runs parallel to Rampart Road approximately 250 yards to the south. All buildings south of Rampart Road (including the CDC) connect to SM1. The second sewer main (SM2) is a 12" pipe running parallel to Rampart Road approximately 200 yards to the north. SM2 services the majority of existing development north of Rampart Road and south of Laporte Avenue. It should be noted that there are existing, but minimal (1-5), connections just north of Laporte Avenue which contribute wastewater to SM2 (Figure 2.1).

In addition to the two major sewer mains previously discussed, two additional connections are made to the City of Fort Collins sewer system from the Foothills Campus. To the south of SM1 and SM2, there is a sewer main which services Hughes Stadium and Chrisman Field. To the north of SM1 and SM2, there is an 8" VCP pipe which services several existing buildings including the Agricultural Engineering Research Center near the north east property boundary of the Foothills Campus. These mains have not been included in this planning study because of their location and proximity to SM1 and SM2. However, wastewater from these two mains could be captured and routed to an onsite treatment facility if so desired and proper infrastructure is put into place.

2.1.4 Wastewater Availability and Characterization

Existing Foothills Campus buildings are constructed with combined plumbing; that is, all sources of wastewater are combined into one pipe. One building on the Foothills Campus, Atmospheric Chemistry, contains separated plumbing. Separated plumbing enables wastewater with high treatment requirements to be handled separately from wastewater with lower treatment requirements. This increases overall efficiencies of treatment processes and encourages reuse of the valuable components contained in wastewater. Three categories of Foothills Campus wastewater have been identified as labwater, blackwater and graywater.

As its name denotes, labwater can be defined as wastewater from laboratory processes such as dishwashing, hydraulic experimentation, autoclave cooling and so on. Labwater can vary greatly in composition depending on its source. However, labwater is generally low in organic and pathogenic contaminants. For the purpose of this analysis, the assumption is made that existing buildings will not be retrofitted with separated plumbing. This is because retrofitting building plumbing adds potentially inhibitive expenses. CSU, however, has the option to separate wastewater plumbing in proposed development. Separation of plumbing in proposed development provides an opportunity to maximize efficiency of treatment as well as maximize the efficiency of extraction of valuable resources found in wastewater such as biogas and reusable irrigation water.

2.1.4.1 Labwater

Labwater is broadly defined as any wastewater originating in a laboratory. Complexity of varying laboratory functions, equipment and procedures in different research buildings on the Foothills Campus produces labwater of varying qualities. For the purposes of this report, Foothills Campus labwater is divided into two distinct categories: lab sink water and lab process water.

Lab Sink Water: All wastewater originating within a laboratory that is washed down laboratory sinks. This includes glassware wash water, non-hazardous waste from experimentation and other sources which may potentially contain shock loads of chemical or biological constituents.

Lab Process Water: Wastewater originating within a laboratory which is used for hydraulic experimentation (e.g. ERC hydraulics laboratory), cooling (e.g. autoclaves), or other processes with minimal to no contact with chemical or biological constituents. Flow rates of lab process water can be expected to be considerably higher than lab sink water.

Lab sink water has the potential to contain shock loads of chemical or biological contaminants. Shock loadings increase the probability of biological treatment process upset because onsite systems have lesser dilution ratios as compared to larger municipal collection systems. Two options for treatment of lab sink water include (1) appropriate treatment of lab sink water prior to introduction into biological treatment processes, and (2) discharge of lab sink water offsite for treatment. Because lab process water is expected to be substantially greater in volume than lab sink water, separation of these streams will reduce treatment costs whether option (1) or option (2) is chosen. In option (1), the volume of wastewater with higher treatment requirements is reduced to lab sink water, equating to lower treatment costs. In option (2), the amount of treatment necessary to purchase from the City of Fort Collins Utilities is reduced by capturing and treating lab process water onsite.

If lab sink water is treated onsite, shock loadings of chemical or biological contaminants will require appropriate treatment or monitoring. Dilution with combined wastewater from existing buildings is also an option to be considered if volumes are sufficient to prevent treatment process upsets. Monitoring would allow lab sink water to be diverted to a holding tank for advanced treatment only when shock loadings are detected. Monitoring may also allow

CSU to regulate illegal disposal of hazardous waste in laboratory sinks. Both removal and monitoring of these contaminants provides CSU with the opportunity to explore technologies which potentially could play a greater role in the protection of public safety in water systems. Lab process water, by nature, can be considered low in contaminants harmful to treatment processes. Lab process water may be combined with graywater for treatment, or if deemed sufficiently clean may be used for direct reuse. For example, waste ERC hydraulics labwater is currently applied as irrigation water after discharge into College Lake.

2.1.4.2 Blackwater

Blackwater is redefined in contrast to the general definition found in Chapter 1 to depict Foothills Campus specific composition. Foothills Campus blackwater may include wastewater from sources with high organic loading rates including toilets, kitchen sinks, dishwashers, garbage disposals, animal manure, and organic kitchen refuse. This wastewater is highly concentrated with organics and nutrients from which valuable components can be extracted: e.g. methane biogas and nutrients. Blackwater contains the majority of pathogens and emerging contaminants found in wastewater e.g. pharmaceuticals and other endocrine disrupting compounds. Concentrating these constituents which require high levels of treatment into a reduced volume waste stream allows for an overall more efficient treatment system.

2.1.4.3 Graywater

Graywater is redefined in contrast to the general definition provided in Chapter 1 to reflect anticipated sources specific to the Foothills Campus. Foothills Campus graywater will mainly consist of wastewater generated in hand wash sinks but may contain other was waters as found in future development including showers or clothes washers. As compared to blackwater, treatment required for reuse of graywater is considerably less intensive. Graywater can be treated using simple technologies such as wetlands, which have many benefits outside of treating graywater. Wetlands are typically low maintenance, aesthetically pleasing, provide habitat for wild animals and sequester carbon dioxide from the atmosphere. Graywater treated through wetland systems may be easily reused in drip irrigation systems or for toilet flushing.

2.1.5 Treatment Process Footprint

The required footprint for treatment processes has been estimated to ensure allocation of space at suitable locations on the Foothills Campus property. Footprints were estimated assuming: wetland treatment for labwater and graywater, anaerobic treatment for blackwater.

2.1.5.1 Wetland Treatment

Wetland treatment systems are plant based systems in which photodegradation, biodegradation, sedimentation, and plant uptake processes all serve as mechanisms to remove contaminants from wastewater. Two types of wetland designs are typically used, free water surface (FWS) and subsurface flow (SSF). As the description denotes, FWS wetlands are similar to a pond with free standing water containing plant species such as cattail. SSF wetlands contain a bed of gravel or other porous material through which water flows. Plants in SSF wetlands rely on nutrients from wastewater for growth as there is typically no soil involved.

Wetlands are capable of providing advanced treatment and nutrient removal for low solids and organic containing wastewaters such as labwater and graywater. Wetlands are very low in required maintenance, although sediment removal is necessary every several years (depending on solids loadings). At this time sediment and plant material must be removed and the wetland is replanted. In addition to treating graywater, wetland treatment could be utilized for effluent liquid from a blackwater treatment process. Greater removal of pathogens, metals, solids, and other contaminants would result, providing higher quality water for end use, e.g. irrigation.

2.1.5.2 Anaerobic Digestion

Anaerobic digestion is a biological wastewater treatment process which relies on the absence of oxygen to breakdown complex organics and destroy pathogenic contaminants in wastewater. Anaerobic digestion is widely used in wastewater treatment and is very reliable for high levels of pathogen and solids destruction. Anaerobic digestion has been recognized as a superior technology for sustainable wastewater treatment due to its ability to capture useful byproducts (i.e. biogas and nutrients) from wastewater during treatment. Biogas is one important product resulting in anaerobic digestion and can contain up to 80% methane. Methane biogas is a sustainable resource which can be used for several purposes depending on the volume and site specific cost effectiveness including:

- Direct use in boiler steam generation, incorporation of a heat exchanger allows reactor to be heated to required digestion temperature.
- Electricity generated using a turbine or engine can be sold back to the utility during peak hours.
- After high levels of polishing, methane can be sold to the natural gas utility.

Costs saved or earned using one of these end uses for methane typically outweigh the cost of energy input to run the anaerobic reactor. For this reason, anaerobic digestion can offset operational costs or even create a positive flow of income for the operator.

2.1.5.3 Maximum Probable Footprint

Anaerobic digestion of blackwater and constructed wetland treatment of graywater were chosen for wastewater management planning purposes because CSU is seriously considering them and in conjunction, are expected to require the maximum probable footprint, therefore reserving sufficient space within the Foothills Campus property. In addition, these treatment methods are “green technologies” and require very low energy input. Labwater and GW flows have been combined to calculate wetland footprint to provide sufficient sizing if determined that labwater streams are to be routed for wetland treatment.

Wastewater generation rates are calculated based off maximum Foothills Campus buildout. Total buildout conditions are modeled after available existing and proposed development data available from the CSU Department of Facilities Management. The 2005 report and corresponding calculations prepared for facilities titled "Water Evaluation of Colorado State Universities Foothills Research Campus" was also used as a reference for footprint calculations (Criswell & Roesner 2005).

Efforts were made to size treatment footprints conservatively large to ensure that sufficient room is available for planning purposes. A further, more detailed study using GIS databases to determine proposed building usage and existing building wastewater generation rates by building usage type should provide better estimation projected footprint sizing. Sizing includes flows from proposed and existing buildings. Flows from the CDC existing and proposed development are included in sizing. Although not entirely probable that CDC development will be captured, the intention for this study is to develop a plan that has the potential to capture maximum possible wastewater generated.

2.1.6 Modular Treatment Systems

Each development scenario within this report contains a schematic flow diagram showing "treatment modules expandable with development." Modular treatment systems are designed to be easily expandable with development and easily upgraded to treat specific waste streams. Such systems could be predesigned 'package' or 'modular' plants provided by a third party, or in-house designed modular systems tailored specifically to meet Foothills Campus wastewater treatment needs.

Given that Foothills Campus development will occur in phases, wastewater treatment demands will increase as the campus is developed. Modular treatment systems provide CSU with the opportunity to add treatment capacity as wastewater treatment demand increases.

Using modular treatment reduces the complexity of expansion and allows capital investment to be spread over time and in conjunction with the pace of development. This strategy avoids construction of oversized treatment units. Modular systems also allow CSU to implement new or more effective technologies which are available when expansion is needed. Because of their relatively small footprint and reduced complexity of construction and installation, planning efforts are facilitated using modular treatment systems.

As an example, say that CSU has installed an operating modular treatment system on the Foothills Campus with enough capacity to treat existing development wastewater. We will refer to this as 'Plant A'. CSU has expected to expand Plant A with treatment modules as development proceeds to planned buildout. Three new buildings are constructed along the edge of the foothills at the maximum distance from Plant A. These buildings are expected to produce exceptionally higher volumes of graywater and labwater than planned for in conveyance infrastructure with typical blackwater generation rates. Capacity in conveyance infrastructure is available to carry blackwater and some graywater/labwater. Fortunately, CSU has not invested capital upfront in Plant A. Therefore, capital is available to construct a wetland or other biological treatment system close by these buildings to treat graywater and labwater.

2.1.7 Development Phases

Three distinct phases of development are referenced within this report including Existing, Phase I and Phase II. Existing refers to buildings included in the 2008-2009 water allocation spreadsheet provided by the CSU Department of Facilities Management. This spreadsheet uses water flow from 2007 to allocate water for the 2008-2009 year. Phase I of development refers to Foothills Campus development which is projected to occur within the next 10 years. Phase II of development refers to Foothills Campus development which is projected to occur within the next 20+ years. For the purpose of this report, and because more detailed information was not available, it has been assumed that all Phase I and Phase II development will happen at a steady rate over the duration of each phase. It was assumed that Phase I would run from 2008-2017 and Phase II would run from 2017-2032.

2.1.8 Foothills Campus Wastewater Demand Estimations

To determine the results presented within this report, in depth calculations were necessary to understand existing wastewater demands and characteristics and extrapolate these characteristics to proposed development. The results of this report are based off these calculations. To calculate many of the values presented within this report, it was necessary to make assumptions. It is necessary to understand these assumptions when reading the results of this report. A detailed description of calculation methods and assumptions is provided in Section 2.2 of this document.

2.1.9 Flood Plain Boundaries

Generally, development of wastewater treatment facilities within the 100 year floodplain is restricted for purposes of public safety. A 100 year floodplain is the footprint of water resulting from a flood which is expected to occur an average of once every 100 years (e.g. 1% chance annually). For purposes of locating onsite wastewater treatment facilities at the Foothills Campus, available flood plain maps were first consulted. Two sources of flood plain maps were examined for the Foothills Campus property: FEMA flood insurance rate maps (FIRM) and Fort Collins City floodplain maps. Because the Foothills Campus is outside of the City of Fort Collins City Limits and 'Growth Management Area,' no flood plain mapping is provided for this region. For this reason, this report will rely on information from FEMA FIRM maps. According to FEMA, FIRM map panels 08069C0960F and 08069C0957F, the entire Foothills Campus is located out of the 100 year floodplain (Zone X classification). However, by viewing Fort Collins City floodplain maps it is evident that some portion of the Foothills Campus along the Pleasant Valley and Lake Canal may be within a 100 year floodplain. If additional floodplain data is available for the Foothills Campus, it is recommended that further investigation is undertaken to determine flooding risk within proposed areas of process construction.

2.1.10 Other Factors to Consider

2.1.10.1 Energy Production

Biogas is a potential output for treatment from Foothills Campus wastewater. Extraction and capture of biogas from wastewater produced onsite allows direct reuse for generation of steam in boilers. If biogas production rates are sufficiently high, electricity can be generated.

2.1.10.2 Additional Waste Streams

Opportunity exists to incorporate various other waste streams into the Foothills Campus treatment system. Two waste streams readily available at the Foothills Campus are ‘Manure Mountain’ and a continuous source of animal waste generated at various locations. Accepting food waste from buildings is also an option. If difficulties arise with the main campus composting operation, food waste from the main campus could be trucked to the Foothills Campus for treatment in anaerobic digesters. Each of these streams increase loading of anaerobic treatment systems in turn increasing the volume of biogas created and subsequent energy production.

2.1.10.3 Toilet Design

Low flow toilets and vacuum toilets provide a significant reduction in potable water demands. Dual flush toilets are also available. In a dual flush system the user has an option to flush liquid waste with smaller volume of flushwater or solid waste with a higher volume of flushwater. Toilets can also be flushed with treated graywater, further reducing potable water demands.

2.1.10.4Vacuum Toilets and Conveyance

Vacuum toilets and conveyance infrastructure are innovative technologies which could be applied for capture of blackwater. Vacuum toilets and conveyance can substantially reduce the amount of water necessary for flushing and conveyance, and effectively homogenize blackwater. Reduced volumes from water savings allow large reductions in infrastructure and treatment process sizing.

2.1.10.5Urine Separation

Urine is a typically sterile, low volume wastewater stream containing a large percentage of nutrients found in wastewater. High nutrient loads and low treatment requirements make reuse of urine as an inexpensive source of fertilizer in agriculture attractive. Urine separation can be accomplished using urine separating toilet technologies.

2.2 Methods

Methods used for estimation of wastewater production, treatment footprint, and methane production from anaerobic digestion are presented in this section.

2.2.1 Existing Building Wastewater Demands

Foothills campus annual water demands for 2007-2008 were made available by the CSU Department of Facilities Management. Actual water demands were provided for sub-metered buildings and calculated water demands were made available for a majority of remaining Foothills Campus buildings. Total Foothills Campus water usage, measured by Foothills Campus master meter, was adjusted across all buildings for which sub-metered data or calculated data was available. Because a number of buildings were not accounted for in sub-metered and calculated water use values, water usage for these buildings is represented by the difference between Foothills Campus master meter usage and sub-metered and calculated usage. It was assumed that all water used was recovered as wastewater. Water losses were likely minimal.

Therefore, only buildings with calculated or sub-metered flows were used to determine existing wastewater production.

Specific building usages for proposed development (e.g. office, research, classroom, etc.) were not available. Therefore, it was not possible to extrapolate wastewater flows from existing building usages to proposed development based on building usage type. To better capture qualities of existing flows, several development regions were created to encompass buildings which should have similar wastewater production characteristics. These regions were chosen to define areas of development for which proposed buildings could be expected to have similar blackwater, graywater and labwater production as existing buildings within the same development regions (Figures 2.3-2.5). Areas in blue represent either buildings which do not produce wastewater e.g. KCSU or buildings hooked into a wastewater line not studied within this report.

Figure 2.3: Northern development regions (legend applies to Figures 2.3-2.5)

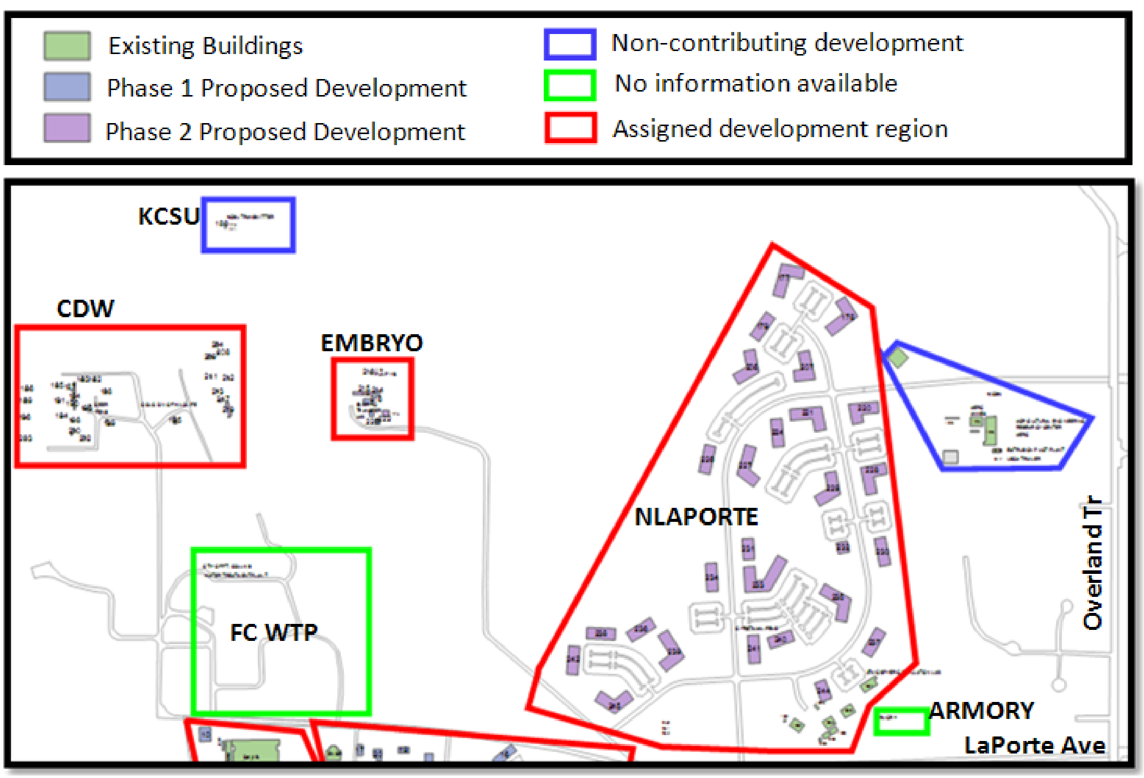


Figure 2.4: Central development regions

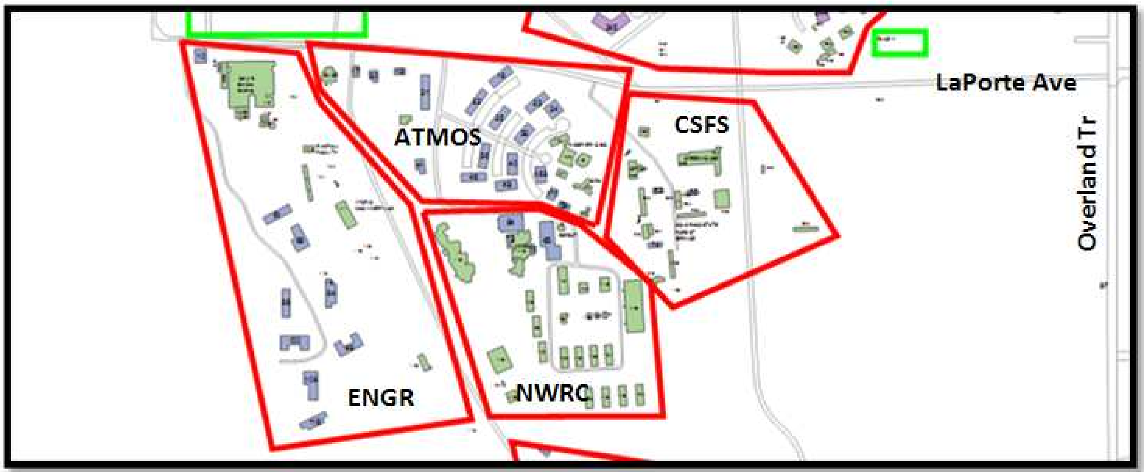
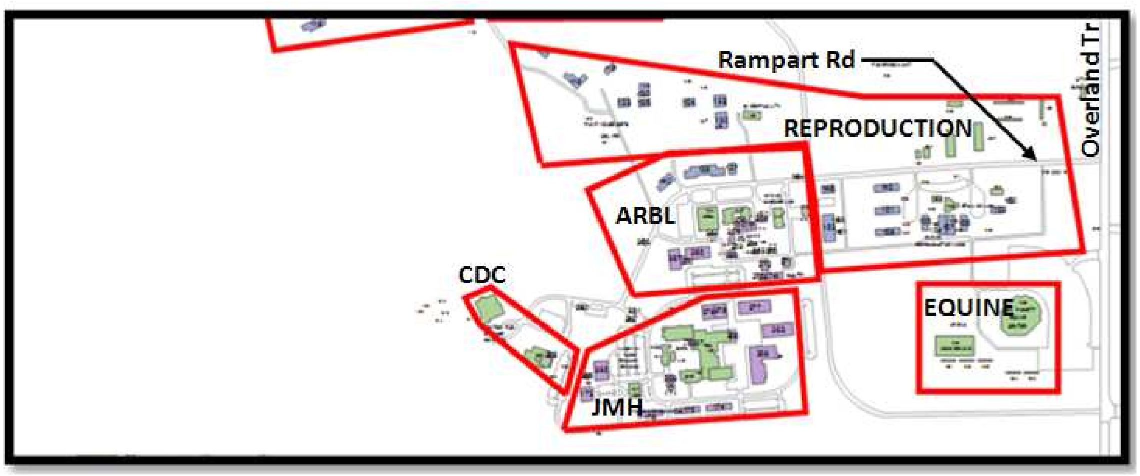


Figure 2.5: Southern development regions



Wastewater production within each development region was calculated separately for blackwater, graywater and labwater. To determine existing development wastewater generation rates, it was first necessary to calculate populations within each development region. Population information was not available; therefore population was predicted individually by building within each development region by assuming a value of gross square feet (GSF) occupied per person. Existing GSF occupied per person was estimated to match existing wastewater flows and building populations. Estimates were based off the amount of blackwater and graywater expected from existing wastewater flows. Two categories of population density were determined for existing buildings, existing population density and maximum population density (Table 2.1). Maximum population density was based off values used in the 2005 "Water Evaluation of Colorado State University's Foothills Research Campus" (Criswell & Roesner 2005).

Table 2.1: Population densities used in wastewater demand calculations

Development Region	Existing Population Density [ASF/Person]	Existing Population	Maximum Population Density [ASF/Person]	Maximum Population
NLAPORTE	2,000	7	500	27
CSFS	1,000	53	500	105
ENGR	1,000	146	500	291
ATMOS	2,000	30	500	120
NWRC	1,000	176	1,000	176
CDC	1,000	37	500	73
REPRODUCTION	1,000	51	500	101
ARBL	1,000	44	500	88
JMH	1,000	113	500	226

Calculated population numbers were used to estimate blackwater and graywater flows for each development region. Blackwater and graywater production values were calculated from modified typical household per capita production values from Boulder and Fort Collins Colorado taken from calculations performed for "Water Evaluation of Colorado State University's Foothills Research Campus" (Criswell & Roesner 2005). Both municipalities break down water use into six categories: toilet, bath/shower, washing machine, faucet dishwasher and leaks (infiltration). Leaks were not included in calculations because decentralized treatment systems are not prone to leak contributions. Only toilet water was included as a contributor to blackwater, while the remaining uses were calculated as contributors to graywater. A percentage of each wastewater generating function was assumed to occur at work during a typical day. Final per capita generation rates were 14.5 GPCD blackwater and 8.0 GPCD graywater (Table 2.2).

Table 2.2: Blackwater and graywater generation rate determination

Fixture	Category	Average Use [GPCD]	% In Office	Average Use In Office [GPCD]		Average BW [GCPD]	Average GW [GCPD]
Toilet	BW	19.3	75%	14.5		14.5	--
Bath/Shower	GW	15.7	0%	0.0		--	0.0
Washing Machine	GW	15.6	0%	0.0		--	0.0
Faucet	GW	10.6	75%	8.0		--	8.0
Dishwasher	GW	2.4	0%	0.0		--	0.0
Leaks	--	6.5	0%	--	**TOTAL:**	**14.5**	**8.0**

Per capita generation rates were then used to calculate the volume of blackwater and graywater produced for existing and maximum population densities. These values have been provided (Table 2.3).

Table 2.3: Calculated blackwater and graywater production values

Development Region	Existing Population	Existing BW Produced [GPD]	Existing GW Produced [GPD]	Maximum Population	Max BW Produced [GPD]	Max GW Produced [GPD]
NLAPORTE	7	101	56	27	390	215
CSFS	53	766	421	105	1,518	835
ENGR	146	2,111	1,161	291	4,207	2,313
ATMOS	30	434	239	120	1,735	954
NWRC	176	2,544	1,399	176	2,544	1,399
CDC	37	535	294	73	1,055	580
REPRODUCTION	51	737	405	101	1,460	803
ARBL	44	636	350	88	1,272	700
JMH	113	1,634	898	226	3,267	1,797

Both existing and maximum blackwater and graywater production values were used in further calculations. Maximum blackwater and graywater production were used to determine maximum probable production rates in GPD/GSF for development region. The maximum probable production rates were then used to predict blackwater and graywater generation from proposed development. Maximum blackwater and graywater production values were also used for process sizing calculations, because they represent the existing buildings at predicted capacity.

Existing blackwater and graywater generation were solely used for calculation of labwater production rates. Labwater production rates were calculated using the existing population (Table 2.3), GSF of existing buildings and calculated wastewater flows from existing buildings. Total combined wastewater production (WW_{total}), blackwater production and graywater production were calculated for individual buildings within each development region. Labwater generation rate was calculated by:

$$LW\ [GPD/GSF] = \frac{WW_{total}\ [GPD] - BW\ [GPD] - \ GW\ [GPD]}{GSF\ of\ building}$$

Using this method of labwater calculation, labwater does not increase with population increase. However, for prediction purposes labwater generation rates calculated for maximum building populations in existing and proposed development has been increased by a factor of 10%. This factor is intended to account for increased lab use from increased building populations over time. Generation rates for blackwater, graywater and labwater have been estimated as previously discussed (Table 2.4). Blackwater, graywater, and labwater generation from proposed development have been estimated using these values.

Table 2.4: Blackwater, graywater and labwater generation rates

Development Region	BW/GSF [GAL/FT^2]	GW/GSF [GAL/FT^2]	LW/GSF [GAL/FT^2]
NLAPORTE	0.021	0.011	0.010
CSFS	0.020	0.011	0.177
ENGR	0.020	0.011	0.229
ATMOS	0.020	0.011	0.017
NWRC	0.010	0.006	0.105
CDC	0.020	0.011	0.695
REPRODUCTION	0.020	0.011	0.088
ARBL	0.020	0.011	0.024
JMH	0.020	0.011	0.300

The area of proposed buildings was available as proposed building footprint area provided by the CSU Department of Facilities Management. Further details were not accessible; however an assumption of two floors per building was suggested by the CSU Department of

Facilities Management GIS specialist. Therefore gross square footages of proposed development have been calculated by multiplying building footprint areas by two. Wastewater generation was then calculated for proposed buildings within development regions using blackwater, graywater and labwater generation rates (Table 2.4).

Wastewater discharge from the City of Fort Collins Utilities Water Treatment Facility and the National Guard Armory, both contributing to SM2 along Laporte Avenue, was not included in flow calculation and plant sizing because this information was not available. The City of Fort Collins Utilities plans to measure wastewater flow from their treatment facility in the near future and will provide CSU with results. Calculations should be updated when this information becomes available.

2.2.2 Anaerobic Reactor and Constructed Wetland Sizing

Footprint calculations were performed for treatment wetlands and anaerobic digestion processes to estimate land area needed. Anaerobic process sizing was performed following design guidelines found in literature (Tchobanoglous et al. 2003) (Grady et al. 1999). Design assumptions include:

- Cylindrical reactor
- 25-foot depth
- 15-day hydraulic retention time (HRT)
- 10% increase in footprint for equipment housing

Reactor volume was calculated by:

$$Reactor\ Volume\ [ft^3] = HRT\ [days] \times Inflow\ [\frac{ft^3}{day}]$$

Surface area necessary for reactor footprint was extracted by:

$$SA\ [ft^2] = \frac{4 \times Reactor\ Volume\ [ft^3]}{\pi \times depth\ [ft]}$$

To calculate wetland footprint surface area, the following assumptions were made:

- Rectangular wetland footprint

- Vertical wetland walls
- Free water surface design
- 3-foot depth
- 14-day hydraulic retention time (HRT)

Wetland surface area was calculated by the following equation:

$$Wetland\ SA\ [ft^2] = \frac{Flow\ [ft^3/day] \times HRT\ [days]}{Depth\ [ft]}$$

2.2.3 Methane Production from Anaerobic Digestion

Methane estimation design equations from (Tchobanoglous et al. 2003) were used to determine theoretical methane production from anaerobic digestion of Foothills Campus blackwater. The following assumptions were made:

- 1850 mg/L chemical oxygen demand (COD) in Foothills Campus blackwater
- 90% COD is biodegradable
- 90% biodegradable COD (bCOD) is destroyed in reactor
- 4 days digester solids retention time (SRT)
- Cell yield coefficient: 0.08 g VSS/g bCOD
- Endogenous coefficient: 0.03 d^{-1}
- Digester gas is 50% CH_4

COD concentration is from laboratory characterization of blackwater from the Atmospheric Science Chemistry building. bCOD concentrations are based off measured volatile solids concentrations, in the reactor demonstration part of this study, influent solids have measured 91% VSS indicating biodegrability of substrate. Greater than 90% VSS destruction was experienced in this study, indicating at least 90% destruction of biodegradable substrate composition. High rate anaerobic reactors are typically operated at long SRTs, a value of four days provides an estimation at the lower end of probable SRT. Cell yield and endogenous coefficients were based on literature values (Tchobanoglous et al. 2003). Digester gas from blackwater digestion is typically above 50% and has been measured as high as 76% (Wendland et al. 2007) (Kujawa-Roeleveld 2005).

Calculation of methane estimates was performed by:

$$(bCOD_i - bCOD_f)\ [kg/m^3] \times Q[m^3/d] - 1.42 \times P_x[kg/d]$$

Where:

- Q = blackwater inflow rate
- P_x = Net cell mass

The value of \$9.02 / ft^3 used to determine methane value was taken from the average of 2005-2008 Colorado natural gas pricing index found at the National Energy Information Administration website (EIA 2009). Direct supplementation of methane with natural gas on an equal heating value basis was assumed for valuation purposes.

2.3 Foothills Campus Treatment Scenarios

Four distinct wastewater treatment scenarios have been developed to provide onsite treatment of Foothills Campus wastewater. Each scenario presents varying locations of treatment processes and captures varying portions of development. All of the provided scenarios will allow CSU to maximize the efficiency of valuable resource extraction from the Foothills Campus wastewater, e.g. nutrients, reusable water and biogas, increasing the sustainability of the Foothills Campus.

Three of the four scenarios are considered 'zero discharge.' In zero discharge scenarios, all wastewater is captured for treatment onsite within the Foothills Campus property. Although, some exceptions may exist, e.g. lab sink water as discussed further in this report. The four scenarios are as follows:

- S_I – Complete Onsite Cluster Treatment **(Zero Discharge)**
- S_{II} – Onsite Cluster Treatment for Proposed Development Only
- S_{III} – Onsite Cluster Treatment for Proposed Development; Onsite Centralized Treatment for Existing Development **(Zero Discharge)**
- S_{IV} – Complete Onsite Centralized Treatment **(Zero Discharge)**

2.3.1 Scenario I – Complete Onsite Cluster Treatment

In S_I, wastewater from the Foothills Campus is captured and treated by a series of 'onsite cluster' treatment facilities. An 'onsite cluster' treatment facility is described as:

> A wastewater treatment facility located to capture and treat wastewater flow from a specific cluster of development within a larger property. Clusters of development are defined within a larger property using identifiers such as regions with high density of development, using favorable topography, and following along lines of existing infrastructure.

S_I is a zero discharge scenario. Therefore in S_I, all Foothills Campus wastewater is treated within the Foothills Campus property boundary. In S_I, proposed buildings may be plumbed to provide separate or combined wastewater streams. A potential wastewater flow schematic for S_I separates plumbing into labwater, graywater and blackwater streams in proposed development (Figure 2.6).

In this schematic, a treatment process is incorporated to separate wastewater from existing development into streams similar to blackwater and graywater which can be combined with proposed development labwater, graywater and blackwater streams for treatment, resource extraction and eventual reuse. Incorporation of modular treatment systems provides CSU flexibility in initial capital expenditure as well as future treatment process location.

Although lab sink water is shown to leave the Foothills Campus property, flows from lab sinks have been included in process sizing calculations to ensure adequate space for treatment systems. The decision to send lab sink water offsite will be a tactical planning decision made by CSU. As previously discussed, lab sink water has potential to contain shock loadings of constituents which may have a malicious effect on biological treatment systems if not monitored for or pretreated prior to introduction to onsite treatment systems.

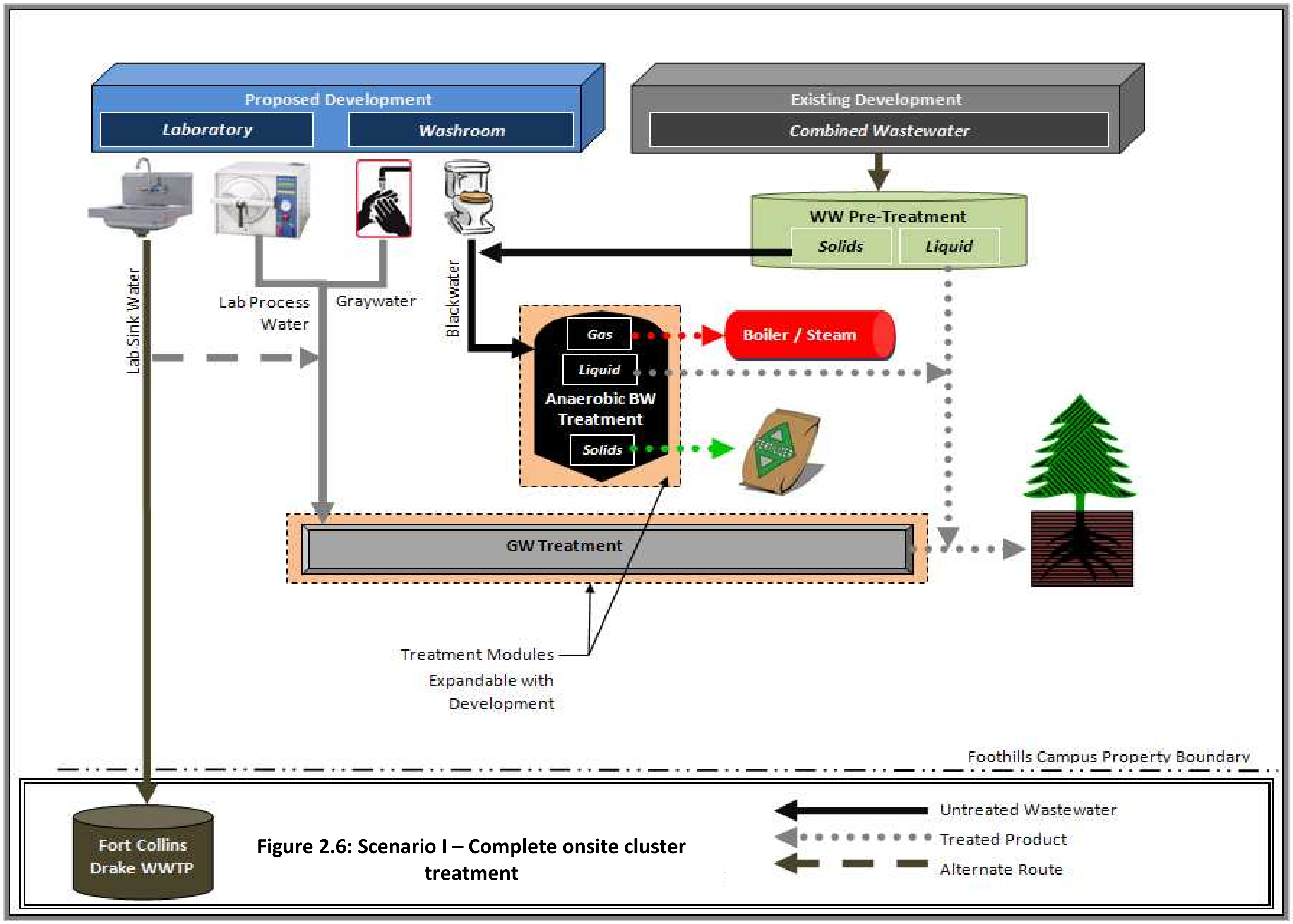
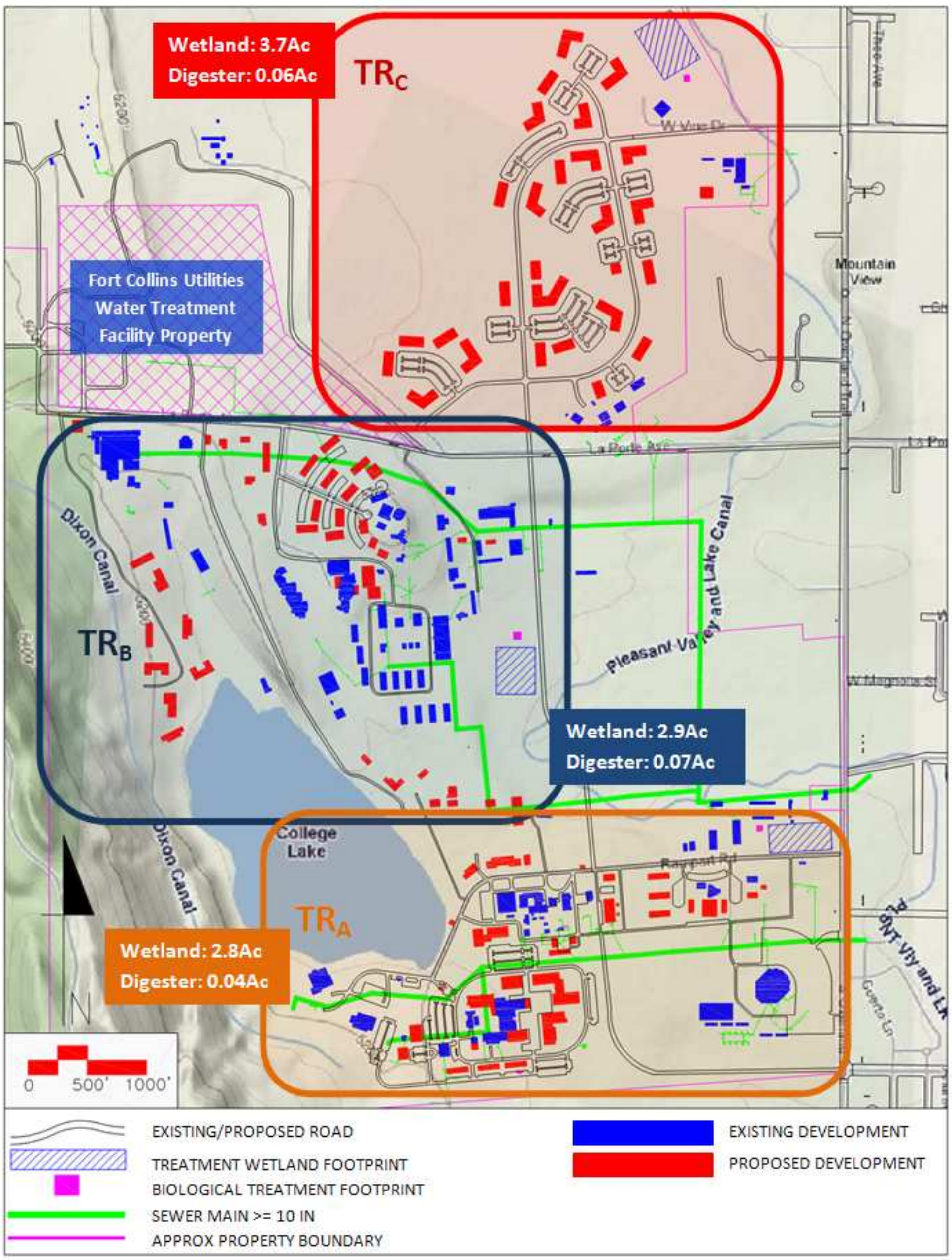


Figure 2.6: Scenario I – Complete onsite cluster treatment

2.3.1.1 Site Layout

Three clusters of development have been identified on the Foothills Campus based on existing topography, location of existing infrastructure, and location of existing and proposed development. Three main clusters of development have been identified as TR_A, TR_B and TR_C as detailed in the introduction of this report (Figure 2.7).

Figure 2.7: Potential S_I layout of treatment facilities



Potential cluster treatment plant locations are chosen within each treatment region. Similar to treatment regions, plant locations are chosen according to topography, existing infrastructure location and ability to capture both future and existing development. One cluster treatment plant is located within each treatment region. Maximum probable treatment process footprints are shown to scale (Figure 2.7).

2.3.1.2 Discussion

Advantages (Conceptual)

- Small-scale, onsite wastewater treatment is a practice gaining popularity as existing wastewater infrastructure deteriorates and becomes strained with new development. Application at CSU will bring attention to the University's commitment to innovative and sustainable technology.

- Operation of such facilities provides CSU with the opportunity to implement various advanced technologies for onsite wastewater treatment, placing CSU on the forefront of innovation and providing research opportunities.

- Through elimination of resource export, zero discharge fulfills the aim of sustainability.

Advantages (Operational)

- Capture of methane biogas provides CSU with a sustainable source of energy produced onsite.

- Nutrient enhanced anaerobic digester effluent can be used for irrigation of non-consumptive crops, such as the Colorado State Forest Service pine beetle kill pine tree replacement crop.

- CSU will capture much of the nitrogen, phosphorus, and organic matter in their wastewater and apply it for positive uses locally, saving the fraction of these materials which would otherwise return to receiving water bodies to encourage environmental problems including eutrofication.

- Source separation reduces the volume of expensive treatment facility capacity required for wastewater by diverting low contaminant risk graywater for simpler less costly treatment.

- Flexibility: cluster systems allow capacity to be added as needed and not invested in upfront 20 years before it is necessary.

- Redundancy: if an upset occurs at one facility, flow may be rerouted to another cluster facility.

- Locating treatment facilities near the wastewater producers allow the water to be reused near the source without the need for extraneous piping and energy inputs. Placing wastewater treatment, reuse, and capture near developments may prove educational and beneficial to conservation measures.

Disadvantages

- Some rerouting of existing sewer mains and laterals may be necessary to reach cluster treatment facilities. Lift stations may be necessary. Unless existing buildings are retrofitted with separated plumbing, flows from existing infrastructure will be combined, requiring a treatment process to (1) separate into blackwater, graywater and labwater similar quality flow streams or (2) treat combined wastewater separately.

- Sporadic shock loads of toxic and pathogenic materials from lab sink water must be (1) treated onsite with advanced monitoring and pre-treatment technologies or (2) sent offsite for treatment.

- Multiple facilities likely will increase capital costs, O&M requirements and security obligations.

- Permitting of these 'different from the norm' facilities may be rigorous.

- Monitoring of multiple facilities will add cost and require increased maintenance.

2.3.2 Scenario II – Onsite Cluster Treatment for Proposed Development Only

In S_{II}, a series of 'onsite cluster' treatment facilities are constructed to capture and treat wastewater from proposed development only. Wastewater from existing development continues to flow offsite for treatment. As provided in S_I, an 'onsite cluster' treatment facility is described as:

> A wastewater treatment facility appropriately located to capture and treat wastewater flow from a specific cluster of development within a larger property. Clusters of development can be defined within a property using identifiers such as development densities, topography, and existing infrastructure.

S_{II} is not a zero discharge scenario; as wastewater from existing buildings is exported offsite for treatment. CSU will be required to purchase treatment capacity from the City of Fort Collins Utilities for treatment of exported wastewater. In S_{II}, proposed buildings may be plumbed to provide separate or combined wastewater streams. For a potential wastewater flow schematic for S_{II}, proposed development plumbing is separated into labwater, graywater and blackwater streams (Figure 2.8). Separate treatment of labwater, graywater and blackwater facilitate resource extraction and eventual reuse. Modular treatment systems allow treatment capacity expansion with increases in development.

Although lab sink water is shown as leaving the Foothills Campus property, flows from lab sinks have been included in process sizing calculations. The decision to send lab sink water offsite will be a tactical planning decision made by CSU. As previously discussed, lab sink water has potential to contain shock loadings of constituents which may result in process upsets to biological treatment systems if not monitored for or pretreated prior to introduction to onsite treatment systems.

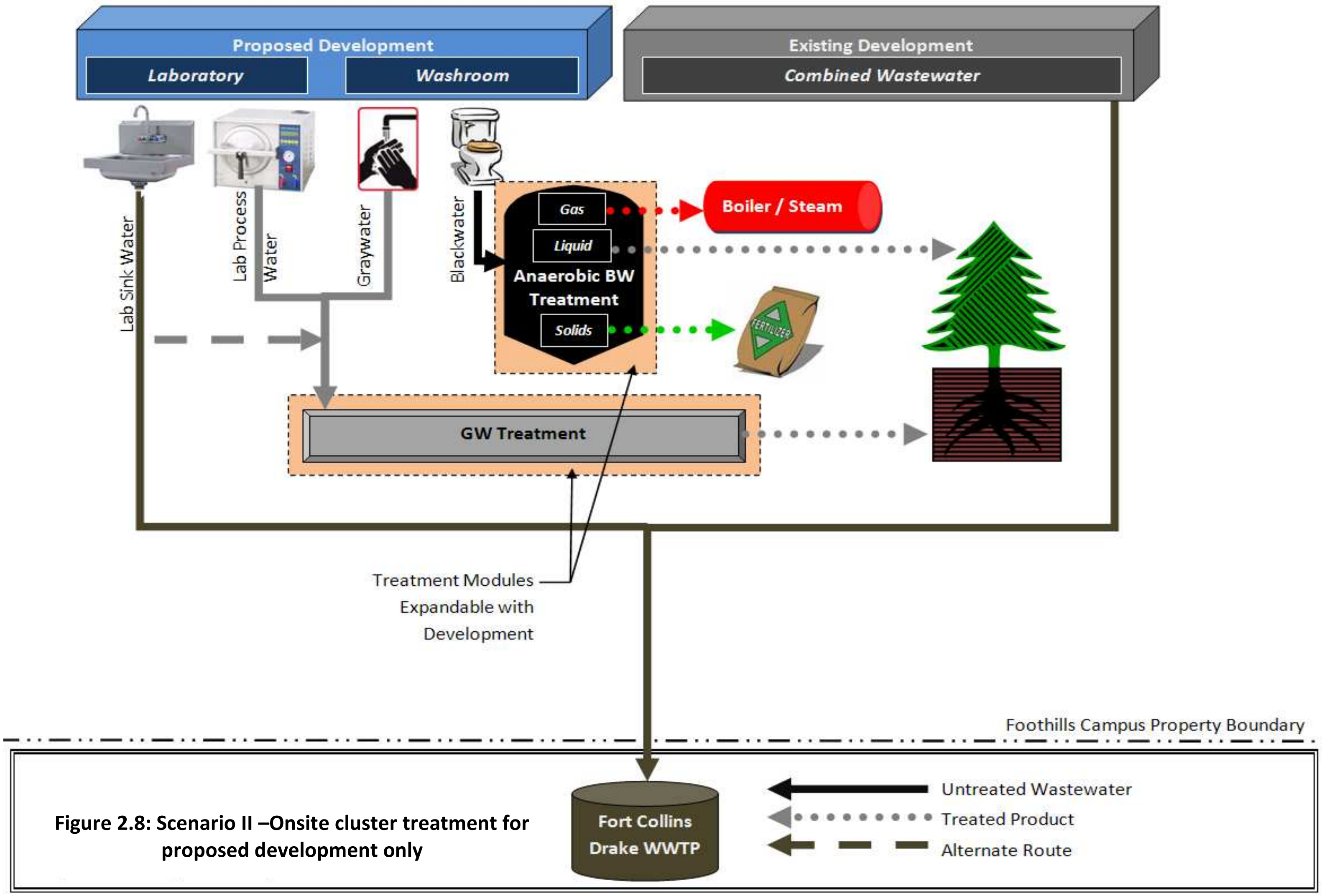
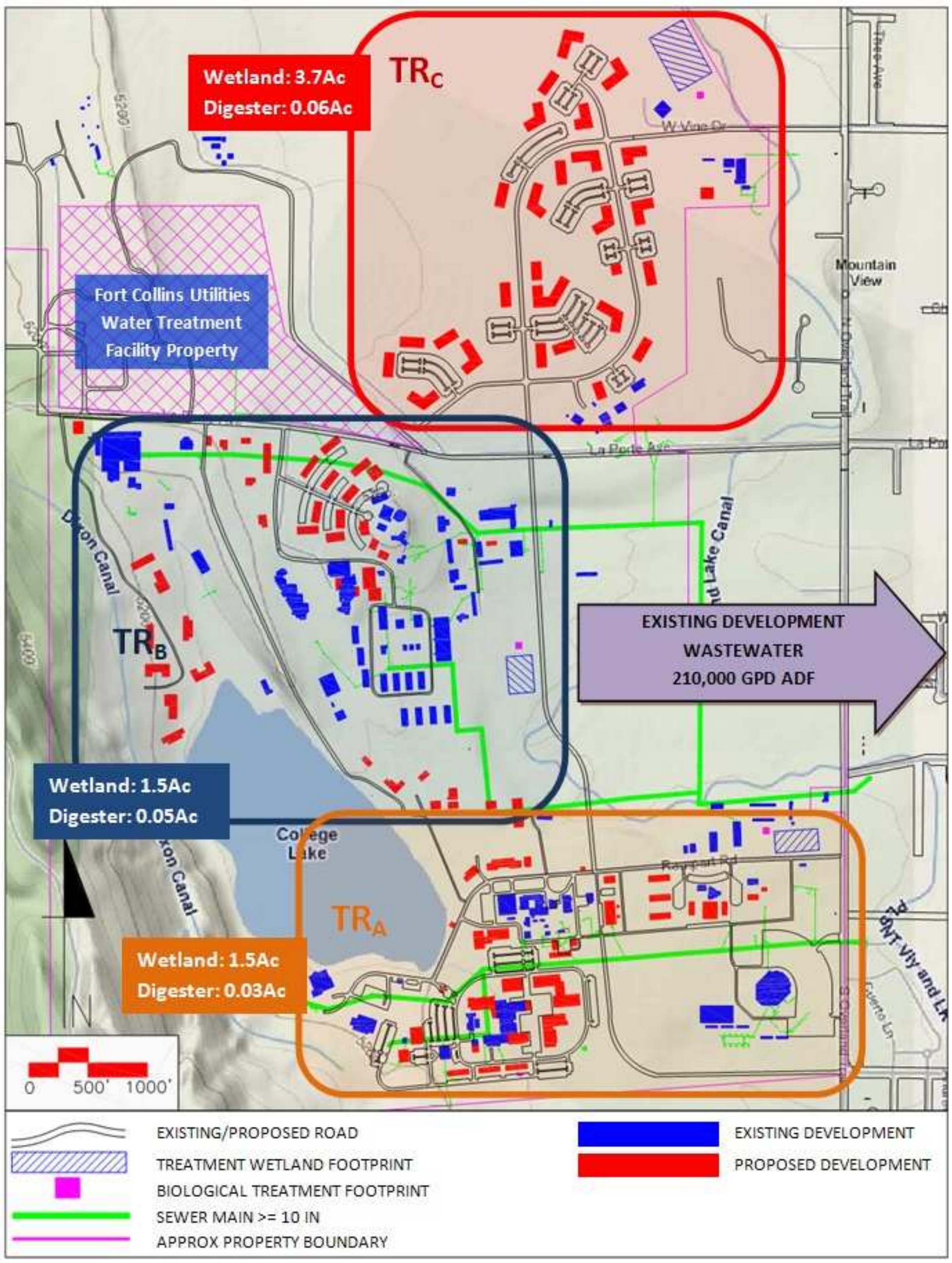


Figure 2.8: Scenario II –Onsite cluster treatment for proposed development only

2.3.2.1 Site Layout

In S_{II}, infrastructure servicing existing buildings is left unaltered. As a tactical planning decision, lab sink water may be incorporated into the existing infrastructure for treatment offsite. Proposed development is serviced by three treatment regions TR_A, TR_B and TR_C (Figure 2.9).

Figure 2.9: Potential S_{II} layout of treatment facilities



Treatment plants are located within each treatment region and required treatment footprint is reduced as compared to S_I because existing development wastewater is treated offsite. In $S_{II,}$ TR_A, TR_B and TR_C service the same development regions as discussed in S_I. However, in S_{II} existing development is not captured by onsite systems and is sent offsite for treatment. Potential cluster treatment plant locations and sizes are provided within each treatment region. Because S_{II} leaves existing infrastructure unaltered, S_{II} could be used as a starting point to eventually lead into operation of S_I. For example, CSU could initially install modular treatment systems to capture proposed development and incorporate existing buildings into the onsite treatment capacity at a later time, mimicking S_I.

2.3.2.2 Discussion

Advantages

- SII maintains all advantages of cluster treatment discussed in Scenario SI.
- Treating solely proposed development reduces initial capital investment.
- Lab sink water from proposed development can be combined with existing wastewater and be sent offsite for dilution in the municipal conveyance system, reducing the chance of potential toxic or shock loadings of contaminants which could upset onsite biological treatment processes where dilution is not as available.
- Because existing wastewater is sent offsite, it is not necessary to deal with potential toxic or pathogenic shock loadings in existing combined wastewater.
- Proposed development allows for the separation of wastewater flows and subsequently permits tailored cluster treatment processes for each wastewater type, greatly increasing efficiency of treatment and resource utilization.

Disadvantages

- Water is exported offsite, which does not fully maintain independence of Foothills Campus wastewater treatment.
- Because existing development is sent offsite, onsite reusable water and energy generation are reduced.
- Multiple facilities likely will increase capital costs, O&M requirements and security obligations.

2.3.3 Scenario III – Onsite Cluster Treatment for Proposed Development; Onsite Centralized Treatment for Existing Development

S_{III} is a hybrid between onsite cluster treatment and onsite centralized treatment. In S_{III}, wastewater from existing development is captured and treated at an onsite centralized facility. Wastewater from proposed development is captured and treated by onsite cluster treatment facilities which can be added as development occurs. As previously described, cluster treatment facilities are defined as:

> A wastewater treatment facility appropriately located to capture and treat wastewater flow from a specific cluster of development within a larger property. Clusters of development can be defined within a property using identifiers such as development densities, topography, and existing infrastructure.

S_{III} is a zero discharge scenario, incorporating four treatment systems. Three treatment systems serve proposed development by treatment region and one treatment system serves all existing development. In S_{III}, proposed buildings may be plumbed to provide separate or combined wastewater streams. In a potential wastewater flow schematic for S_{III}, plumbing is separated into labwater, graywater and blackwater streams in proposed development (Figure 2.10). In this scenario, the onsite centralized treatment process provides similar resource extraction and eventual reuse prospects. Modular treatment systems are also incorporated for treatment of wastewater from proposed development, providing flexibility in initial capital expenditure as well as treatment process location.

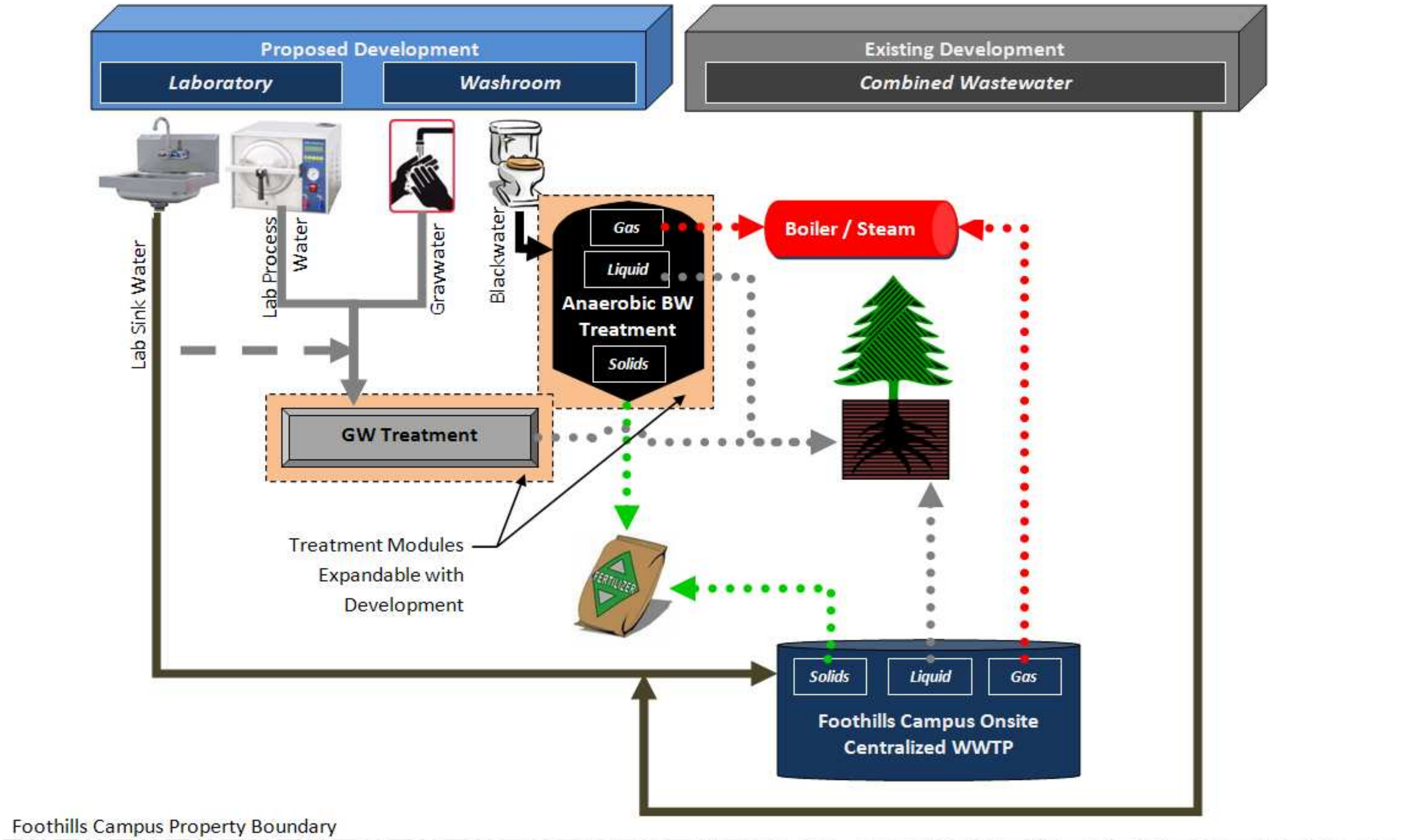


Figure 2.10: Scenario III – Onsite cluster treatment for new development; onsite centralized treatment for existing development

Untreated Wastewater
Treated Product
Alternate Route

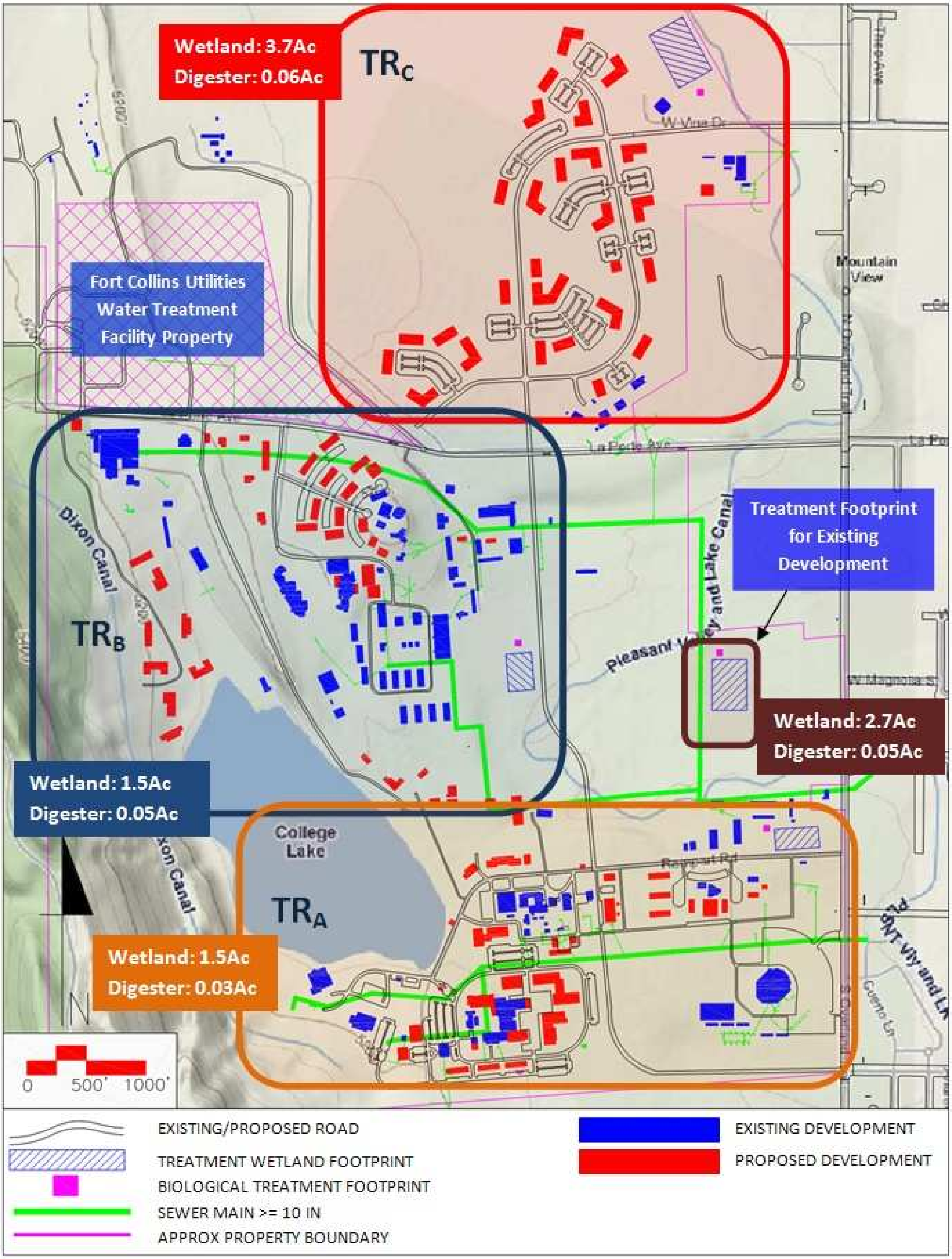
Lab sink water from proposed development is shown to combine with wastewater from existing buildings. Process sizing calculations do not account for this combination, as lab sink water is incorporated in with proposed development sizing calculations. Assuming lab sink water is 25% of total labwater, labwater from proposed buildings would account for 1/3 of flows into the centralized wastewater treatment plant. Assuming this 1/3 total flow would be transported to a wetland for treatment for sizing purposes, the treatment footprint would increase by 1.6 acres. Assuming this 1/3 of flow would travel to an anaerobic digester, the treatment footprint would increase by 0.3 acres. Although another treatment method would most likely be incorporated, these numbers are offered for planning purposes.

2.3.3.1 Site Layout

In S_{III}, infrastructure servicing existing buildings can be utilized to route wastewater to the onsite centralized treatment facility. This facility has been located to utilize existing infrastructure. This location is somewhat elevated above Pleasant Valley and Lake Canal, located off Overland Trail near the confluence of SM1 and SM2. As shown in the S_{III} schematic, lab sink water may be routed to the onsite centralized facility if dilution is sufficient, or if properly handled it may be routed for onsite decentralized treatment.

Proposed development is serviced by three treatment regions TR_A, TR_B and TR_C (Figure 2.11). Potential cluster treatment plant locations are provided within each treatment region. Similar to treatment regions, plant locations are chosen according to topography, existing infrastructure location and ability to capture both future and existing development as provided by the CSU Department of Facilities Management. One cluster treatment plant is located within each treatment region. Maximum probable treatment process footprints have been provided.

Figure 2.11: Potential S_{III} layout of treatment facilities



2.3.3.2 Discussion

Advantages

- Onsite cluster treatment facilities maintain all advantages mentioned in S_I.
- Because the onsite centralized treatment facility to treat wastewater from existing development can be aligned with existing conveyance infrastructure, existing development wastewater does not need to be majorly rerouted.
- Enhanced treatment processes and/or monitoring to manage toxic and pathogenic materials from lab sinks can be used in the centralized treatment facility.
- Lab sink water from proposed development can be routed to the centralized facility, thus diluting wastewater with potential to upset biological treatment processes and eliminating the need for such enhanced treatment processes and/or monitoring at cluster facilities.
- Similar to Scenario S_{II}, cluster facilities can be designed smaller and specifically for separate wastewater streams.
- Zero discharge fulfills the aim of sustainability by eliminating resource export.

Disadvantages

- Requires the construction of an extra facility in addition to cluster facilities.
- All disadvantages previously mentioned for onsite cluster systems remain the same.

2.3.4 Scenario IV – Complete Onsite Centralized Treatment (Zero Discharge)

In S_{IV}, all Foothills Campus wastewater will be captured and treated in an 'onsite centralized' facility. An 'onsite centralized' facility is described as:

> A single treatment facility located within the property boundary of the Foothills Campus, appropriately located to capture and treat all wastewater generated within the Foothills Campus. All wastewater from both proposed and existing development will be routed to this facility. Process design will maximize valuable resource recovery from wastewater. Water reuse and biogas generation are likely process outputs.

S_{IV} is a zero discharge scenario. Therefore, all wastewater from the Foothills Campus will be treated within the Foothills Campus property boundary. Centralized onsite treatment provides CSU the ability to utilize existing Foothills Campus conveyance infrastructure. If wastewater from proposed buildings is combined, both proposed and existing development can utilize the same conveyance infrastructure. However, capacity available in existing lines may limit the amount of proposed development that can link into existing lines.

A flow schematic (Figure 2.12) shows wastewater from proposed development separated into labwater, graywater, and blackwater streams for treatment in modular treatment systems. As development increases, modular treatment systems can be added to service additional development. Separate plumbing (e.g. labwater, graywater, and blackwater) increases the efficiency of treatment systems as well as valuable resource extraction (e.g. nutrients, reusable water, and biogas).

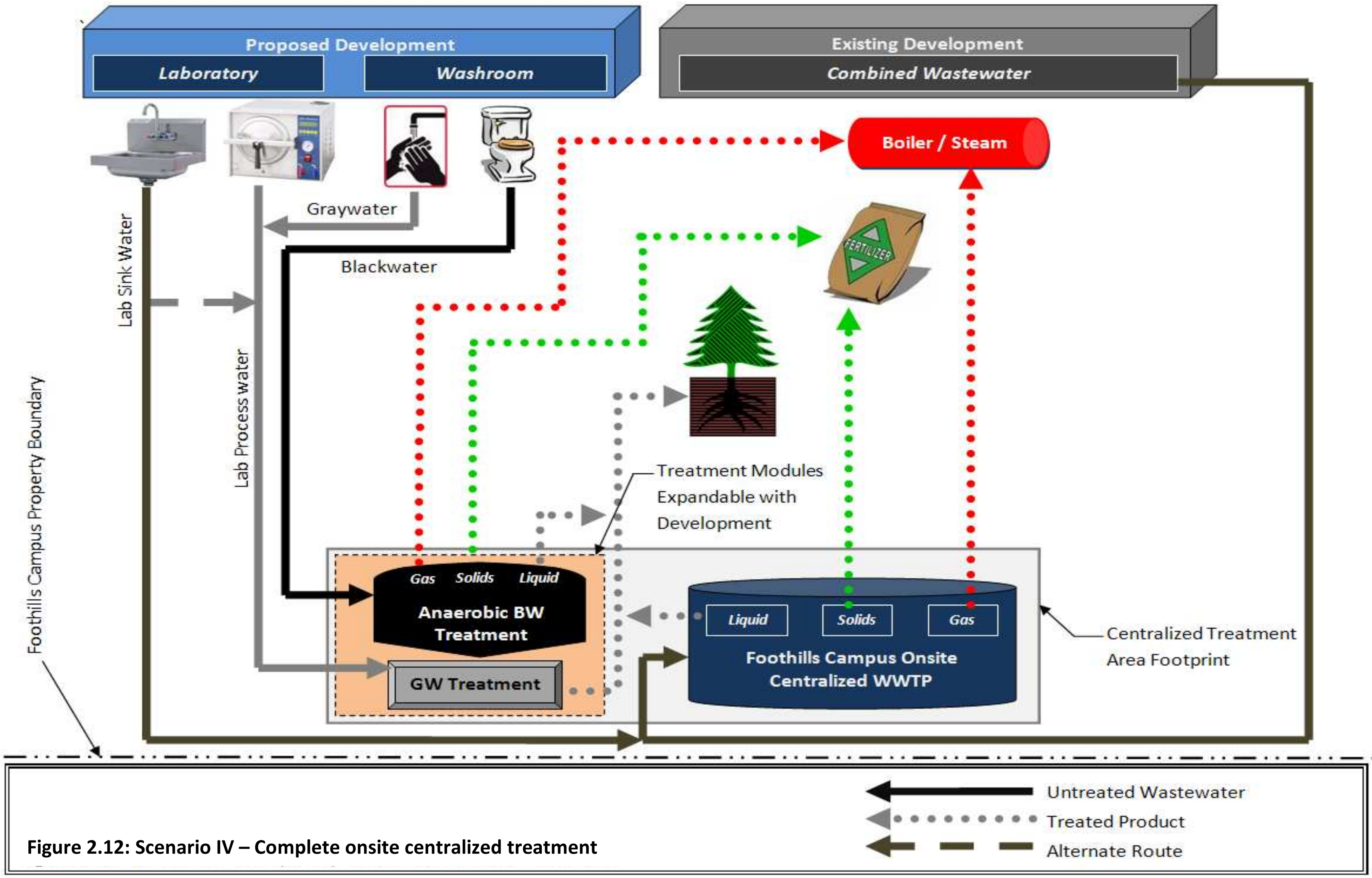
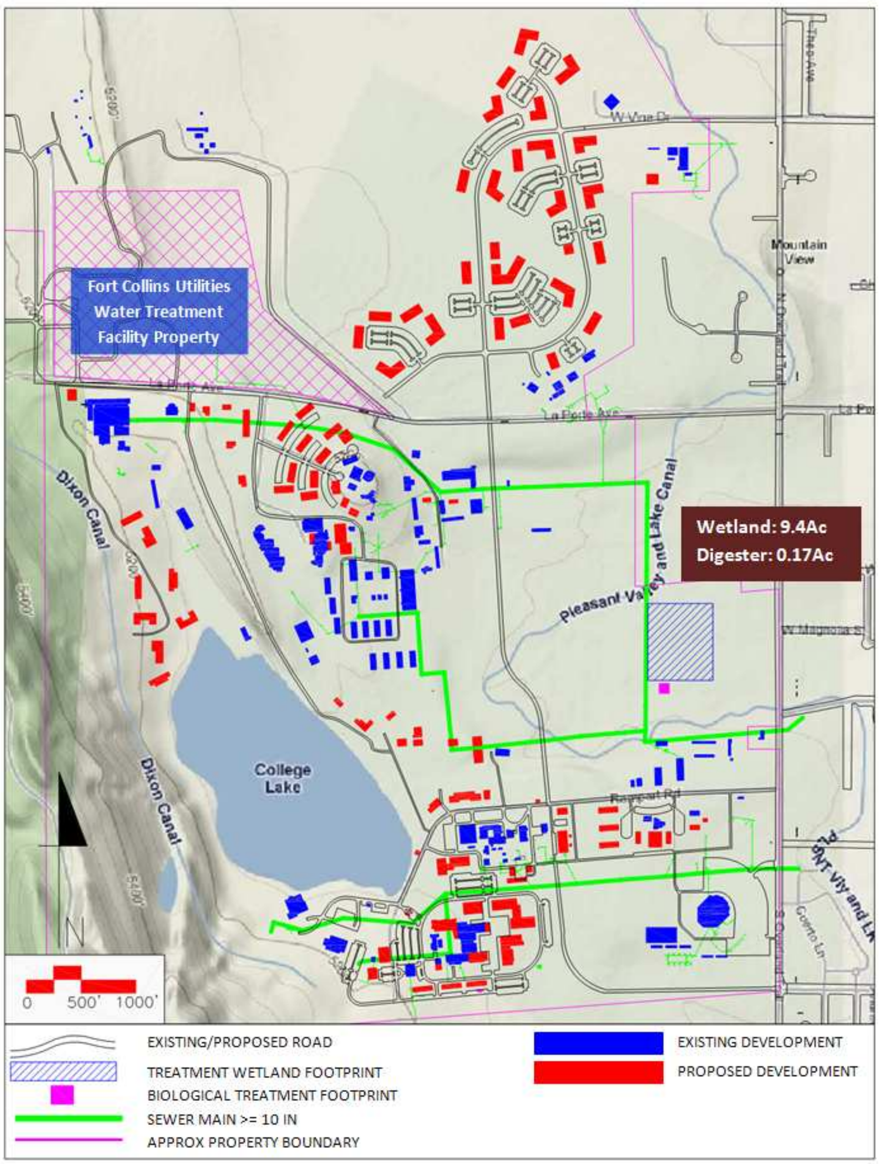


Figure 2.12: Scenario IV – Complete onsite centralized treatment

2.3.4.1 Site Layout

A single treatment facility will be located such that all proposed and existing development can be served. A likely location for this facility is the low point just north of Rampart Road and just west of Overland Trail to either side of the Pleasant Valley and Lake Canal along the central eastern fringe of the Foothills Campus (Figure 2.13). This location has been chosen for two reasons: favorable topography and effective utilization of existing Foothills Campus conveyance infrastructure. Maximum probable footprint has been allocated for S_{IV}.

Figure 2.13: Potential S_{IV} layout of treatment facilities



2.3.4.2 Discussion

Advantages

- A single facility will likely require less capital investment, and could potentially require less O&M and security requirements depending on many factors including treatment technologies used.
- Existing infrastructure can be used to transport wastewater to the centralized facility if sufficient capacity is available.
- Toxicity issues associated with lab sink water are potentially reduced due to increased dilution and installation of enhanced treatment processes and/or monitoring systems at the onsite centralized facility.
- Zero discharge fulfills the aim of sustainability by eliminating resource export.

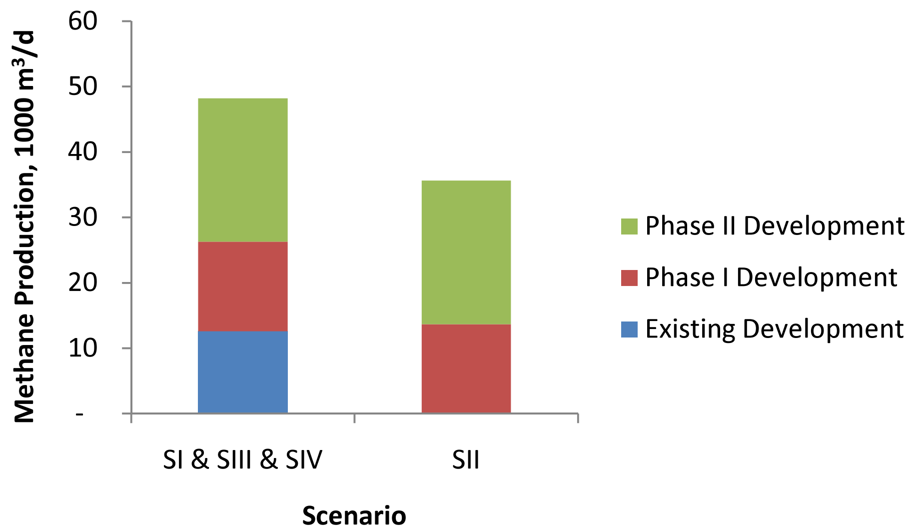
Disadvantages

- Employs fewer innovative management practices.
- If treatment of separated waste streams at the centralized facility is desired, this will require multiple, separate sewers to carry the effluents from the sources to the facility increasing capital infrastructure costs.
- Reused water and energy will require further transportation from the centralized facility back to the location of the wastewater producers, likely requiring greater pumping and energy inputs.
- Large institutional facilities, biosolids handling, equipment operation, and odor issues produce negative impacts to property and surrounding communities.

2.4 Methane Production Potential

Theoretical methane production has been estimated for blackwater flowrates from each phase of development for each development scenario. Figure 2.14 shows theoretical methane production for each scenario. Scenarios one, three and four have been combined because total Foothills Campus blackwater flows are the same for these three scenarios. Scenario II has been separated because this scenario does not capture existing wastewater onsite for treatment.

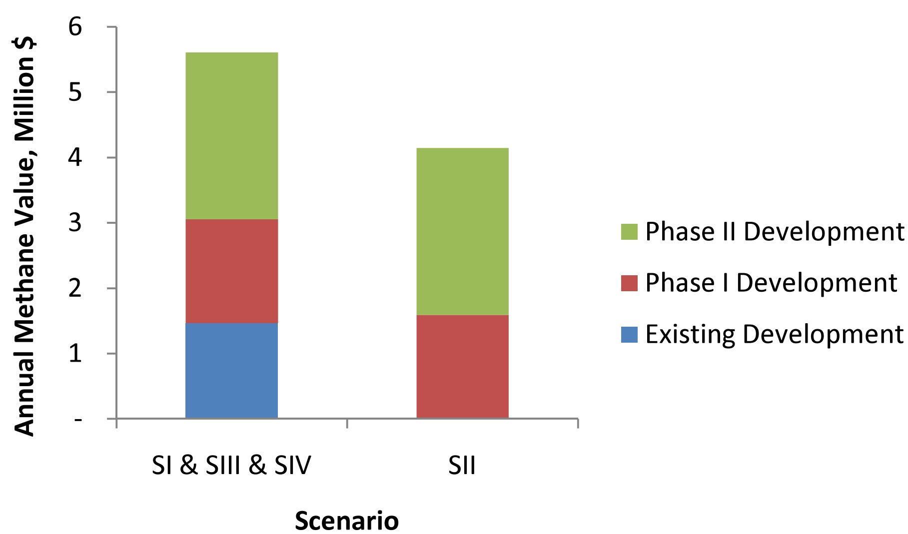
Figure 2.14: Quantity of methane produced from Foothills Campus blackwater digestion



The value of methane produced was calculated (Figure 2.15). This value is calculated based on the following assumptions.

- Direct supplement of natural gas using CH_4 produced onsite
- Value of \$9.02/ft^3 natural gas cost

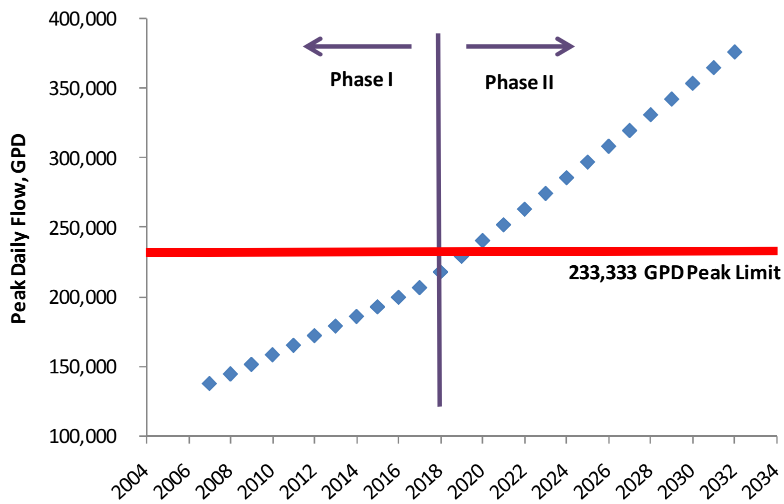
Figure 2.15: Value of methane from Foothills Campus blackwater digestion



2.5 Foothills Campus Capacity Timeline

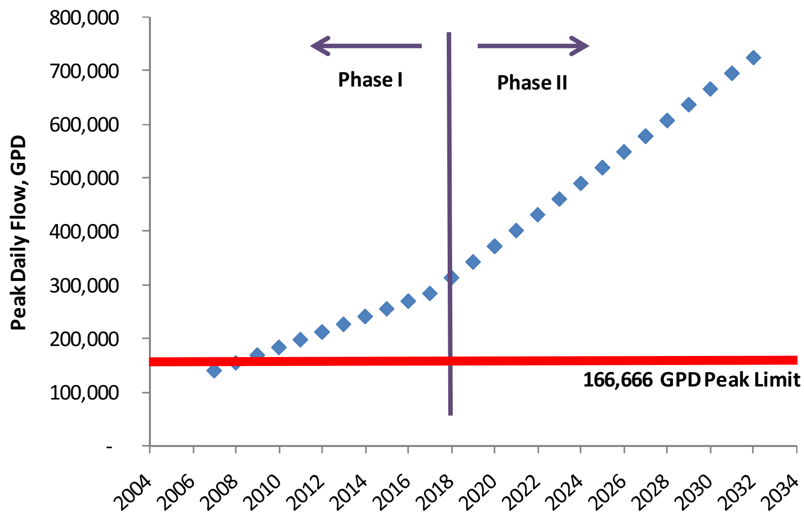
A wastewater discharge limit has been placed on the Foothills Campus by the City of Fort Collins Utilities. There are individual discharge limits for both Foothills Campus sewer mains, SM1 and SM2. SM1 has a peak daily discharge limit of 233,333 GPD and SM2 has a limit of 166,667 GPD. A plot of calculated wastewater flows for SM1 is provided (Figure 2.16). As seen in this plot, the 233,333 GPD peak capacity for SM1 will be reached sometime during the second year of Phase II development.

Figure 2.16: SM1 capacity timeline



A plot of calculated wastewater flows for SM2 is provided (Figure 2.17). According to assumptions, it is expected that the 166,666 GPD peak capacity will be reached in the first year of Phase I development.

Figure 2.17: SM2 capacity timeline



2.6 Conclusions

Developing a plan to provide innovative decentralized wastewater management on the CSU Foothills Campus is a crucial undertaking to avoid future expansion fees imposed on CSU by the City of Fort Collins Utilities. Incorporation of innovative concepts into the plan puts CSU at the forefront of innovation in sustainable wastewater management. Management of wastewater from a research campus setting is inherently complex due to the heterogeneous mix of wastewater sources, for example water form laboratories which has potential to contain biological or chemical contaminants at levels which could provide upset to biological treatment processes and present concerns with human contact in water reuse systems.

Four wastewater management scenarios were developed which incorporate utilization of existing conveyance and collection infrastructure. Options for management of wastewater from existing buildings was considered separately than for new development. Combined plumbing of existing development is maintained in all scenarios to avoid prohibitive modification costs. Three options for management of existing development wastewater were presented. Recommendations for management of wastewater from existing development wastewater were:

- Settle solids, combine solids with proposed development blackwater and untreated liquid with proposed development graywater
- Utilize existing infrastructure to route flow to an 'onsite centralized' treatment facility located on the Foothills Campus
- Continue to route flow through existing infrastructure offsite for treatment

Wastewater from proposed development was separated into four characteristic quality streams in each scenario identified as graywater, blackwater, lab process water, and lab sink water. The treatment technologies considered for graywater and blackwater were constructed wetlands and anaerobic digesters, respectively, to maximize extraction of valuable resources (reusable water, nutrients, and biogas). Lab process water, originating from laboratory processes form which water does not have potential to come in contact with harmful contaminants, e.g. autoclave cooling water, can be combined with graywater for wetland treatment. Lab sink water should be separated due to the potential for this stream to contain shock loadings of chemical or biological contaminants which could upset biological treatment processes and cause concern with human contact in reuse systems. Lab sink water can either be sent offsite through existing infrastructure, or combined with existing wastewater for treatment onsite. If treated onsite, appropriate pretreatment or innovative monitoring systems are recommended for lab sink water to prevent shock contaminant loadings.

CSU will expand the total square footage of Foothills Campus buildings from near 1.0 MSF to roughly 3.5 MSF at complete buildout over approximately 25 years. The value of methane produced from proposed development blackwater at buildout was estimated above $4 million if all biogas produced was used as a direct supplement for natural gas. Flows from existing development on the Foothills Campus are reaching the discharge limit set by the City of Fort Collins.

Incorporating onsite wastewater management architecture to capture proposed development, similar to Scenario II presented in this study, provides a solid starting point to remove the Foothills Campus from the municipal wastewater system. Eventual transition to capture and treat existing development, similar to Scenario I, will allow CSU to completely remove the Foothills Campus from the municipal wastewater system and be completely self reliant. Scenarios I & II in presented, or modified form to capture lab sink water, provide the most advantageous opportunity for CSU to incorporate innovative and sustainable concepts, while providing the option for redundancy if cluster treatment facilities are connected.

As development continues, alternative approaches to management of wastewater from the Foothills Campus will be necessary to prevent additional fees for expansion of Fort Collins municipal infrastructure. By incorporating discussed ADWM concepts for treatment of wastewater on the Foothills Campus, CSU becomes a model for other institutions to follow, creates great potential for future research opportunities examining the future of wastewater management, and contributes greatly to increasing environmental sustainability of campus operations.

3.0 SELECTION OF APPROPRIATE TECHNOLOGY FOR DECENTRALIZED TREATMENT OF BLACKWATER AT FOOTHILLS CAMPUS

3.1 Objective

Numerous high rate anaerobic reactor designs exist for digestion of organic waste streams. The objective of reactor technology selection was to choose the most appropriate treatment technology for use Foothills Campus blackwater. Application of the selected technology in a blackwater digestion demonstration project is thoroughly described in Chapter 4. Selection of the most suitable reactor design was important because this study was intended to provide background performance data on the viability of anaerobic digestion of Foothills Campus blackwater, to be be used in future planning of decentralized wastewater treatment architecture for the Foothills Campus. The goal of the selection step was to determine the reactor technology capable of providing the most efficient treatment of Foothills Campus blackwater, require relatively simple operation and maintenance, and produce methane to contribute to offset of reactor energy inputs. Reactor selection also considered specific criteria important to the CSU Department of Facilities Management for implementation of decentralized blackwater treatment architecture on the Foothills Campus.

3.2 MCDA Model

To provide a reasonable comparison of available treatment technologies a fundamental multi-criteria decision analysis (MCDA) model was developed. Five anaerobic reactor technologies and one aerobic reactor technology were selected for comparison. Chosen reactor technologies represent reactor designs which have shown successful high rate treatment of high strength wastewaters:

- Anaerobic Sludge Blanket Reactor
- Anaerobic Suspended Bed Reactor
- Anaerobic Filter or Packed Bed Reactor
- Anaerobic Complete Mix Reactor
- Anaerobic Membrane Bioreactor
- Aerobic Lagoon

3.2.1 MCDA Step One: Reactor Technology Ordinal Ranking

Two steps were taken to reach a final decision. The first step involved ordinal ranking of each reactor from one (best) to six (worst) against six criteria. Criteria included treatment efficiency, operational complexity, maintenance required, capital investment, energy input, and energy output (Table 3.1). Criteria represent considerations important to the CSU Department of Facilities Management for development of decentralized blackwater treatment architecture on the Foothills Campus.

Table 3.1: Initial MCDA scoring structure

<table>
<tr><th>Criterion</th><th>Treatment Efficiency</th><th>Operational Complexity</th><th>Maintenance Required</th><th>Capital Investment</th><th>Energy Input</th><th>Energy Output</th></tr>
<tr><td rowspan="4">Sub-Criteria</td><td rowspan="2">Ability to Treat High Solids Waste</td><td>Probability of Upset</td><td>Frequency of Clogging</td><td>Volume Necessary</td><td>Influent Pumping</td><td rowspan="4">Methane Production</td></tr>
<tr><td>Probability of Washout</td><td>Frequency of Sludge Removal</td><td>Material Cost</td><td>Mixing</td></tr>
<tr><td rowspan="2">Ability to Treat Particulate COD</td><td>Complexity of Adjustments</td><td rowspan="2">Frequency of Moving Part Replacement</td><td rowspan="2">Footprint Required</td><td>Recycle Pumping</td></tr>
<tr><td>Complexity of Startup</td><td>Aeration</td></tr>
<tr><td rowspan="2">Scoring</td><td>1: Low Efficiency</td><td>1: High Complexity</td><td>1: High Maintenance</td><td>1: High Cost</td><td>1: High Energy Demand</td><td>1: No</td></tr>
<tr><td>5: High Efficiency</td><td>5: Low Complexity</td><td>5: Low Maintenance</td><td>5: Low Cost</td><td>5: Low Energy Demand</td><td>5: Yes</td></tr>
</table>

Sub-criteria are intended to address important fundamental differences amongst reactor technologies within criterion with respect to reactors specific ability to treat blackwater. Each reactor technology was scored from one (worst) to five (best) for sub-criteria. Reactor technology sub-criteria scores were summed within criteria. Total criteria scores were used to assign ordinal ranking to reactor technologies. The reactor with the largest score received the highest ranking (one) whereas the reactor with the lowest score received the lowest ranking (six). Reactors with identical criteria scores received the identical rankings. For example, the criteria level ranking of capital investment is provided (Table 3.2).

Table 3.2: Example reactor criteria ordinal ranking

Capital Investment	Suspended Bed	Sludge Blanket	Anaerobic Filter	Complete Mix	Anaerobic MBR	Aerobic
Volume Necessary	5	5	5	5	3	3
Footprint Required	5	5	5	3	4	2
Material Cost	3	5	4	3	1	5
TOTAL	13	15	14	11	8	10
RANK	**3**	**1**	**2**	**4**	**6**	**5**

Reactor technologies were assigned rankings accordingly within remaining criteria (Table B.1, Appendix B). Detailed sub-criteria scores and an overview of literature treatment performance data for anaerobic reactors are available as a reference (Appendix B).

3.2.2 MCDA Step Two: Analysis of Criteria Importance to CSU

Discussion with CSU Department of Facilities Management regarding desires and goals for development of decentralized blackwater treatment infrastructure provided insight into the relative importance of chosen criteria. To reflect the relative importance of each criterion, relative weightings were assigned to each criterion. Importance weighting for each criterion was assigned using a scale from one (least important) to ten (most important) based on discussions with CSU department Facilities Management (Table 3.3).

Table 3.3: Criteria importance weighting

Criteria	Importance
Treatment Efficiency	7
Operational Complexity	6
Maintenance Required	3
Capital Investment	8
Energy Input	9
Energy Output	9

Initially, a student version of the decision analysis software Criterium DecisionPlus was used to apply relative weightings to reactor technology criteria rankings. This program applied importance weightings to assigned criteria rankings. Criterium DecisionPlus allows the user to weight criteria relative to one another. Each criterion was weighted between 1-10 according to the level of importance to CSU (Table 3.3). Criterion rankings for each reactor were then input to the model and the model was run to determine the best alternative for application on the Foothills Campus. Using this program, the most appropriate technology was selected as the reactor with the highest composite value after summing all six weighted criteria rankings for each reactor technology.

A data comparability problem was identified with comparing technologies by the sum of weighted criteria, in that each criterion is assumed to be of similar relative value. This may not always be a valid assumption, for example the value of capital costs may be orders of magnitude larger than the value of maintenance required. To correct for inconsistencies in relative criteria values during process selection, this step was slightly altered. Instead of comparing reactor technologies using the sum of weighted criteria rankings, the reactor technology with the highest rankings in criteria most important to CSU Department of Facilities management was selected as the most appropriate technology for demonstration.

According to discussions with CSU Department of Facilities Management, energy and capital investment were the most critical factors for comparison and maintenance required was least critical. CSU has adequate capacity to devote to maintenance of the system. Therefore, maintenance required was not ranked as a highly important criterion. However, CSU would like to implement the system that has foremost, the least capital investment with the greatest amount of energy output and the least energy input. Treatment efficiency was ranked as moderately important, making the assumption that all reactors are capable of achieving regulated discharge levels at a minimum. Operational complexity was weighted in the upper mid-range. This weight was intended to reflect that operational complexity was important because operators will need training to operate a full scale reactor; however high operational complexity was not as important to CSU as capital investment or energy requirements.

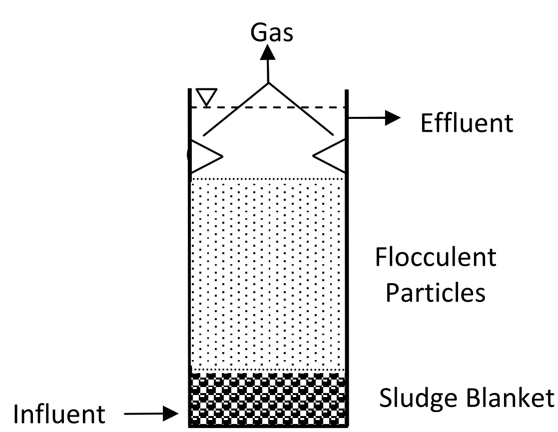
3.3 MCDA Technology Comparison

General reactor design and other important reactor specific considerations are presented for each of the six reactor technologies. Reactor specifics including treatment efficiency and operational complexity are discussed to provide insight into reactor criteria rankings. Literature performance data is provided for reactor technologies as a reference (Appendix B).

3.3.1 Anaerobic Sludge Blanket Reactor (UASB)

In the UASB reactor (Figure 3.1), influent wastewater enters the reactor through a distribution system at the bottom of the reactor and flows upwards within the reactor. Three specific zones of activity can be defined within the UASB. The first zone is at the base of the reactor, where influent wastewater enters. This zone, termed the sludge blanket, is a bed of dense granular or flocculent sludge particles which influent wastewater initially flows through.

Figure 3.1: UASB Reactor



The sludge blanket is where a majority of biological treatment occurs. The second zone is just above the sludge blanket and consists of loose flocculated sludge and less dense wastewater particles. Methane gas, primarily produced in the sludge blanket rises up through the sludge blanket and flocculent zone mixing and allowing increased contact of the sludge particles with influent wastewater.

The third zone, at the top of the reactor, is separated from the second zone by baffles which help settle solids from the third zone returning them for treatment in the flocculent and sludge blanket zones. Clean effluent and methane are extracted within the third zone.

3.3.1.1 Treatment Efficiency

UASB reactors are capable of treating higher organic loadings than other reactors because of the development of a dense granular sludge at the sludge blanket region of the reactor (Tchobanoglous et al. 2003). OLRs between 2-25 kg COD/$m^3 \cdot d$ have been treated successfully in UASB reactors (Grady et al. 1999). UASB treatment efficiencies of 90-95% COD removal have been achieved at 30-35°C at OLRs between 12-20 kg COD/$m^3 \cdot d$ for a variety of wastes (Tchobanoglous et al. 2003). These reactors have functioned at these efficiencies at HRTs as low as 4-8 hours, although it is expected that these low HRTs were achieved on wastewaters which contained mostly soluble COD. UASB treatment of municipal wastewater treatment has yielded a 60% methane biogas concentration (La Motta et al. 2008). For its ability to achieve high solids and particulate COD removal, the treatment efficiency ranking of the UASB ranked second (Table B.1, Appendix B).

3.3.1.2 Operational Complexities

Upflow velocity of influent wastewater must be adjusted to maintain proper suspension and mixing of the sludge blanket region of the reactor. Too high of a velocity will result in a washout of biomass and thus reduced treatment efficiency. Too low of a velocity will minimize the production of methane, and underutilize the capacity of the reactor by limiting biological growth and reducing the contact of influent wastewater and granular sludge by preventing sufficient mixing. Biogas generated within the sludge blanket can substantially contribute to substrate and sludge blanket mixing. Too high levels of gas production within the sludge blanket could encourage washout of biomass. This occurrence is attributed to the uplift ability of rising gas particles produced within the sludge blanket. To ensure mixing across the base of the reactor, influent wastewater must be properly distributed across the base during feed. Implementation of an effective gas separation system and effluent draw-off is also very important to proper functioning of the reactor.

3.3.1.3 Discussion

The UASB is the most popular high rate reactor for treatment of wastewaters since its development in the late 1970's (Hobson & Wheatley 1993). UASB reactors benefit from growth of granular or flocking bacteria in the sludge blanket, which enables the reactor to maintain high levels of treatment without the need for growth media. UASB design allows for separation of SRT from HRT since biosolids remain in the reactor, increasing the rate of treatment and reducing required reactor volumes. In addition, the UASB is relatively simple to construct and does not require a solids settling tank or recycle system, thus saving capital and operation costs as well as reducing required reactor footprint.

The UASB ranked highest in capital investment due to the comparable lack of materials necessary for construction, no recycle infrastructure or media required, and small comparable footprint, the UASB is taller than it is wide unlike the complete mix and does not require spacing for recycle pumping (Table B.1, Appendix B). The UASB also benefits from the lack of moving parts required for reactor operation, resulting in a tie with the suspended bed ranking first for maintenance required.

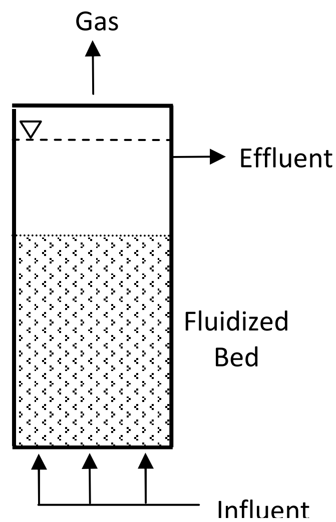
A study comparing treatment efficiency of a UASB reactor to a fluidized bed pre-treating municipal wastewater found more efficient sludge stabilization was achieved in the UASB reactor (La Motta et al. 2008). The UASB also maintained lower energy requirements as the fluidized bed required a high recirculation rate to maintain the fluidized bed material in suspension. Overall removal of TSS and COD was acceptable in the UASB. The UASB tied for first with the anaerobic filter for energy input required (Table B.1, Appendix B). CH_4 concentrations of gas were also comparable to the fluidized bed at 60% (La Motta et al. 2008) (Table B.2, Appendix B).

UASB reactors do have a few disadvantages. The length of the startup period can range between one and six months depending on the type and volume of inoculum used (Souza 1986). Depending on the type of wastewater, it may be difficult to maintain flocculated granules within the reactor. Initially, operational effort may be heavy to ensure loading rate is sufficient to prevent washout of the sludge blanket. Also, UASB reactors do not separate microbial populations responsible for the specific anaerobic digestion stages. Operational complexity of the UASB ranked third against all reactors compared (Table B.1, Appendix B).

3.3.2 Anaerobic Fluidized Bed Reactor

In a fluidized bed, sand, activated carbon, diatomaceous earth, plastics or other similar material is used as growth media for biomass (Figure 3.2). In upflow fluidized bed reactors, operation typically involves high upflow velocities to place all of the bedding material (sand, gravel etc.) into suspension within the liquid portion of the reactor. Often times a recycle system is required to maintain high upflow velocities while maintaining sufficient retention times. Similar to the UASB, fluidized bed depth typically ranges between 4- 6 meters (Tchobanoglous et al. 2003).

Figure 3.2: Fluidized bed reactor



Functionality of a fluidized bed relies on biological growth on suspended media. Under limited effluent recycle, fluidized bed operation is similar to a plug flow reactor, however if recycle is high the fluidized bed flow regime is comparable to a complete mix reactor (Rittmann & McCarty 2001). Choice of support media is essential to the functionality of the reactor and correlates to operational complexity of the reactor. Smaller support media create problems with control of bed expansion and wash out easily, while larger support media can complicate fluidization (Hobson & Wheatley 1993).

In a fluidized bed, solids are removed as they rise to the top of the reactor. The net density of suspended particles (packing) decreases as biomass grows on these particles. Eventually the density of the biomass covered particle falls below that of the surrounding liquid, causing the particle to become buoyant. The influent solids concentration will influence the frequency of solids removal necessary to reduce the amount of solids in effluent. Biomass can be mechanically removed from growth media and media can be returned and reused in the reactor.

3.3.2.1 Treatment Efficiency

In a fluidized bed, COD loading values of 10-20 kg COD/m^3 have been removed up to 90% (Tchobanoglous et al. 2003). Approximately 96% COD removal has been achieved at 35°C at an OLR 10 kg COD/m^3·d for a substrate consisting of mainly glucose at 5000 mg/L COD (Tchobanoglous et al. 2003). Due to limited ability of physical mechanisms in the fluidized bed suspended biomass to trap solids for biodegradation, high removal rates experienced with municipal and industrial wastewaters may not correlate to blackwater substrate. Available literature on the fluidized bed provided studies with mostly soluble industrial or process wastewater as substrate, which are the most suitable wastes for this reactor design. These wastewaters were generally low in solids therefore it is difficult to extrapolate the results of these studies to fluidized bed blackwater treatment. The fluidized bed ranked second for treatment efficiency, alongside the UASB and complete mix reactors for their comparable ability to treat high solids and particulate COD wastewaters (Table B.1, Appendix B).

3.3.2.2 Discussion

The fluidized bed reactor is an attached growth process similar to the anaerobic filter, except that the fluidized bed greatly reduces clogging problems by having suspended media instead of fixed media, for this reason maintenance required for a fluidized bed ranked first, tied with the UASB (Table B.1, Appendix B). In addition, the fluidized bed is relatively simple to construct and does not require a solids settling tank reducing capital costs as well as reducing required reactor footprint, although purchase of media increases costs (capital investment ranked second, Table B.1, Appendix B). Attached growth of bacteria to media allows greater resistance to shock loadings in the reactor feed and also allows the reactor to recover from extended shutdown periods.

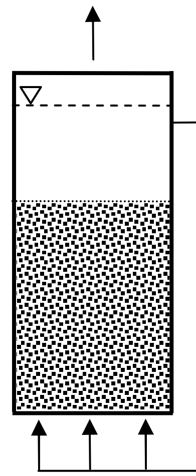
Several disadvantages exist for treatment of blackwater with a fluidized bed reactor. High pumping necessary to maintain media fluidization will greatly increase energy requirements (energy required ranked third, Table B.1, Appendix B). High solids content wastewaters can result in increased maintenance as biomass will accumulate on media quicker and require more frequent removal. Control of bed expansion takes substantial process oversight to prevent washout or insufficient expansion (operational complexity ranked fifth, Table B.1, Appendix B). Turbulence resulting from high upflow velocities reduces the capture of solids from settlement. Thick biomass growth is not commonly observed in fluidized beds because of the elevated upflow velocities used to keep bed expansion high (Tchobanoglous et al. 2003). For these reasons, the fluidized bed is better suited for waste streams containing high percentages of dissolved COD (Tchobanoglous et al. 2003) and not as suitable for high solids wastes such as blackwater.

Startup of the fluidized bed is long and requires high levels of operational oversight to ensure biological growth on media. Change in shape of suspended media due to bacterial colonization also makes control of reactor more challenging (Hobson & Wheatley 1993). Finally, fluidized bed reactors do not separate microbial populations responsible for specific anaerobic digestion stages.

3.3.3 Anaerobic Filter

Anaerobic filter reactors are a type of attached growth anaerobic process with fixed internal media (Figure 3.3). An anaerobic filter can be configured as an upflow or downflow reactor. Between the inlet and outlet ports, there is a section of packed media which is referred to as the filter. The fixed packing can consist of any viable material which biomass will grow on. Typical packing materials include brick, high surface area plastic shapes and tubular plastic.

Figure 3.3: Upflow anaerobic filter reactor



3.3.3.1 Treatment Efficiency

Wastewaters which are readily biodegradable can be treated using anaerobic filter reactors at loadings between 5-10 kg COD/m^3·d and loadings from 1-6 kg COD/m^3·d for high strength wastewaters (Tchobanoglous et al. 2003). COD removal efficiencies at high loading rates are achievable with low particulate waste streams (Table 3.4).

Table 3.4: Selected results from AF performance studies (Tchobanoglous et al. 2003)

Wastewater	Temperature	COD Loading	HRT	COD Removal
	°C	kg COD / m^3·d	hours	%
Citrus	38	1-6	24-144	40-80
Cheese whey	35	5-22	2-8	92-97
Sludge heat-treatment liquor	40	20-30	-	58
Brewery	35	20	1-2	76
Molasses	35	2-13	14-112	56-80
Piggery slurry	35	5-25	0.9-6.0	40-60

Other data shows that anaerobic filters with a 2-10 kg COD/m^3·d loading rate can achieve between 70-80% removal of COD with a retention time of 10-50 hours (Hobson & Wheatley 1993). Because settling is not a critical factor for treatment of solids in the anaerobic filter, the height of the reactor can vary greatly. The anaerobic filter ranked last in treatment efficiency because of its inability to effectively treat high particulate wastewaters such as blackwater without interruptions in treatment capacity.

3.3.3.2 Discussion

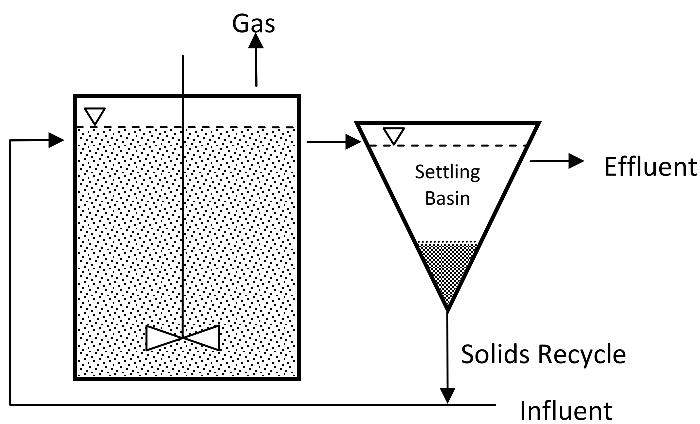
Anaerobic filters have benefits in comparison to other reactors discussed due to the ability of microorganisms to create microhabitats with the reactor system. This enables separation of the microbial populations responsible for specific functions in the anaerobic degradation process, thus increasing the productivity of microbes. Anaerobic filters are also very efficient at treating wastewater containing high levels of COD. Attached growth of bacteria enables greater resistance to shock loadings in the reactor feed, reduced potential for washout from high flowrates, and allows the reactor to recover from extended shutdown periods. Anaerobic filters are relatively simple to construct and there is no need for a solids settling tank or mixing device, thus reducing capital and operational costs, as well as reducing the required footprint. Operational energy input required for treatment in an anaerobic filter consists solely of influent pumping. Therefore, the anaerobic filter was ranked first for energy input (Table B.1, Appendix B).

A major disadvantage of the anaerobic filter for treatment of blackwater, is the potential for clogging the fixed media from high substrate solids concentration. The high solids in blackwater may lead to increased maintenance costs from frequent cleanout of solids (maintenance required ranked third, Table B.1, Appendix B). Main contribution to this ranking was the expected frequency of clogging from high solids wastewaters. The anaerobic filter ranked third for capital investment (Table B.1, Appendix B), as larger reactor volumes are necessary to account for the volume of media used and media can be expensive to purchase, contributing up to 60% of entire reactor cost (Fannin & Biljetina 1987). In addition, proper growth of biomass on attached media typically requires long startup periods. A second place ranking was awarded to the anaerobic filter for operational complexity (Table B.1, Appendix B).

3.3.4 Anaerobic Complete Mix

Complete mix reactors are the simplest form of suspended growth reactors, a large tank with a powered mixer. Mixing within the reactor allows the entire volume of the reactor to be utilized for biomass-substrate contact, maximizing the functional volume of the reactor. The complete mix reactor provides an effluent settling tank and solids recycle in addition to the main reaction tank (Figure 3.4).

Figure 3.4: Complete mix reactor with solids recycle



Provision of a solids recycle allows reduction in both HRT and reactor volume through separation of SRT and HRT. Complete mix reactors are commonly used for large scale municipal treatment works to digest sludge from aerobic processes.

3.3.4.1 Treatment Efficiency

Complete mix reactors are capable of treating wastewaters high in solids concentration at exceptionally high dissolved OLR. Recommended OLRs for complete mix reactors range between 1-5 kg COD/m^3·d with HRTs between 15-30 days (Tchobanoglous et al. 2003). Complete mix reactors with a recycle can operate at similar OLRs between 1-5 kg COD/m^3·d with a great reduction in HRT between 0.5-5 days (Tchobanoglous et al. 2003). The treatment efficiency of the complete mix reactor tied in ranking at second with the UASB and suspended bed reactors for their comparable ability to treat wastewaters high in solids and particulate COD (Table B.1, Appendix B).

3.3.4.2 Discussion

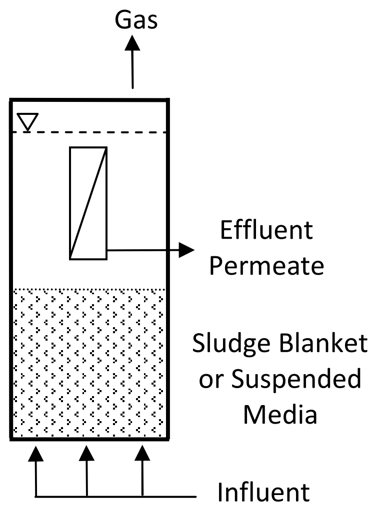
In a complete mix reactor with recycle, 100% of the reactor liquid volume is utilized for biomass and substrate contact. This is much greater than most other reactors incorporating growth media or sludge blankets. Complete mix reactors are well suited to treat wastewater with high solids content and have greater resistance to biological upsets because mixing helps to spread harmful substances (Fannin & Biljetina 1987). In addition, the influent feed design can be extremely simple due to mixing as compared to other types of reactors discussed.

The major disadvantage of the complete mix reactor is the need for a settling tank and effluent recycle to achieve adequate SRT. Addition of a settling tank increases the footprint of the treatment system, increases maintenance and capital costs, and adds complication to operations (Table B.1, Appendix B). Effluent recycle also complicates operations and increases overall capital, maintenance, and operational costs of the system. In the absence of settling and recycling, reactor size must be very high to achieve an SRT high enough for effective conversion of organics to methane. Also, complete mix reactors do not separate microbial populations responsible for the steps involved in anaerobic digestion as is accomplished in anaerobic filter reactors. Overall, the complete mix ranked on the lower end of the spectrum because of the issues discussed (Table B.1, Appendix B).

3.3.5 Anaerobic Membrane Bioreactor (Anaerobic MBR)

The anaerobic MBR can be a hybrid of a UASB or fluidized bed reactor with a membrane filtration step (Figure 3.5).

Figure 3.5: Anaerobic membrane bioreactor



The anaerobic MBR has a zone of biological and substrate contact shown in the figure as the sludge blanket or suspended media. Upper regions of the reactor allow for settling of flocculent solids. In the upper region of the reactor, the concentration of solids is lower relative to the bottom region. A porous membrane is placed in this upper region and effluent permeate is drawn across the membrane surface and out of the reactor.

3.3.5.1 Treatment Efficiency

Performance of the anaerobic MBR will vary depending mostly on membrane pore size. In an anaerobic membrane bioreactor fed blackwater with an average influent COD of 1139 mg/L at a temperature of 37° C, OLR of 2.28 kg COD/m^3·d and a 0.5 day HRT, COD was reduced by an average 86%, although soluble COD in the effluent was still comparatively high (van Voorthuizen et al. 2008). Solids and colloidal reduction was exceptional due to the membrane. The anaerobic MBR ranked first for treatment efficiency due to the incomparable ability of membrane treatment to treat solids and colloidal wastewaters (Table B.1, Appendix B).

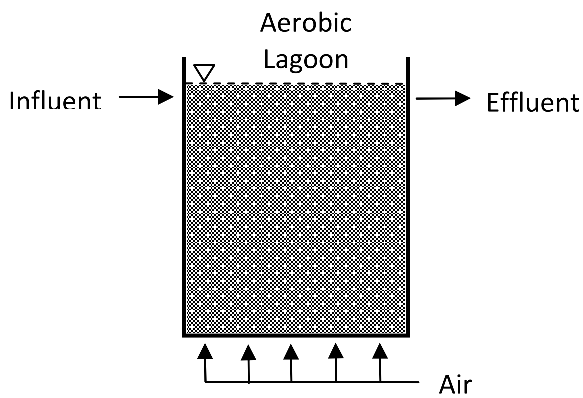
3.3.5.2 Discussion

A major advantage to anaerobic MBR reactors is total capture of solids within the reactor. This enables very high SRT, eliminating washout and increasing treatment efficiency. Also, treatment of solid and colloidal material is near complete. The addition of a membrane, however, adds several disadvantages to those already listed for sludge blanket or suspended bed reactors. Most notable, is the increased maintenance and cost associated with the membrane (maintenance required ranked last, Table B.1, Appendix B). Membranes need to be purchased and replaced periodically. Fouling of the membrane also needs to be addressed when flux across the membrane drops below acceptable values requiring added maintenance effort (operational complexity ranked last, Table B.1, Appendix B). Additional equipment required for membrane operation, additional footprint space necessary to store equipment and the purchase of the membrane resulted in a last place ranking for capital investment (Table B.1, Appendix B). Energy use is also greatly increased to pull the effluent through the membrane (energy input ranked last, Table B.1, Appendix B).

3.3.6 Aerobic Lagoon

For purposes of comparison, a basic aerobic lagoon will be examined. This aerobic technology was chosen for its ability to reach effective treatment with relatively simple construction and operational complexity, desirable traits for application on the Foothills Campus. A basic aerobic reactor consists of a lagoon with an air delivery system (Figure 3.6). Aeration in the lagoon provides mixing and suspension of solids as well as provides oxygen for biological treatment processes.

Figure 3.6: Aerobic lagoon



3.3.6.1 Treatment Efficiency

Typically, aerobic lagoons are not heated and ambient wastewater temperatures are sufficient to maintain sufficient temperature for treatment processes to occur. Soluble BOD is removed quickly in an aerated lagoon and total BOD is removed between 55-80% with an HRT of 4-12 days (Grady et al. 1999). The solids destruction ability of aerobic reactors is somewhat to be desired when compared to most anaerobic digestion reactor designs. For this reason, the treatment efficiency ranking of the aerobic lagoon at fifth came in behind the anaerobic MBR, UASB, suspended bed, and complete mix reactor rankings. The aerobic lagoon did rank higher than the anaerobic filter because less treatment efficiency variances were expected with high solids wastewaters.

3.3.6.2 Discussion

Aerobic lagoons are relatively simple to construct, however a large footprint is required increasing capital investment (capital investment ranked fifth, Table B.1, Appendix B). A lagoon can be an aerated, lined pit with an influent and effluent delivery system. Aerobic treatment systems tend to provide high levels of effluent quality with relatively simple operation (Operational complexity ranked first, Table B.1, Appendix B). Although many benefits exist in aerobic systems, the major drawback compared to anaerobic treatment is operational cost of aeration (energy input ranked third, Table B.1, Appendix B). Aerobic systems require input of energy, whereas anaerobic reactors potentially are energy neutral and can be energy producing. Solids production is also higher in aerobic treatment systems requiring greater disposal costs (maintenance required ranked third, Table B.1, Appendix B).

3.4 MCDA Results

3.4.1 MCDA Step One: Reactor Technology Ordinal Ranking

Ordinal rankings for each reactor technology were assigned based off the sum of sub-criteria scoring results (Table 3.5). Reactor technologies with the same sub-criteria scores were assigned identical rankings. Sub-criteria scores contributing to rankings are provided as a reference (Appendix B).

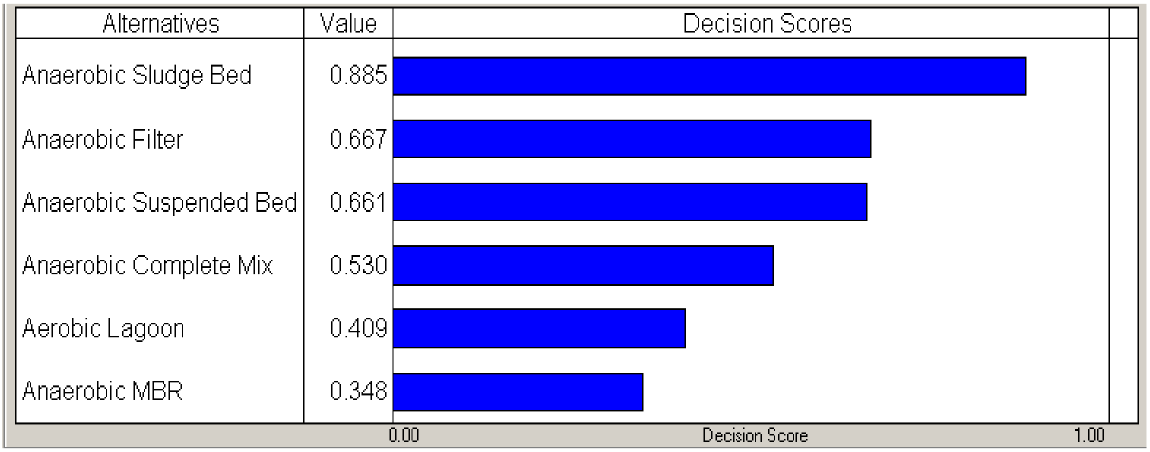
Table 3.5: Reactor technology criteria ordinal rankings

	Suspended Bed	Sludge Blanket	Anaerobic Filter	Complete Mix w/ Recycle	Anaerobic MBR	Aerobic Lagoon
Treatment Efficiency	2	2	6	2	**1**	5
Operational Complexity	5	3	2	4	6	**1**
Maintenance Required	**1**	**1**	3	3	6	3
Capital Investment	2	**1**	3	4	6	5
Energy Input	3	**1**	**1**	5	6	3
Energy Output	**1**	**1**	**1**	**1**	**1**	6

3.4.2 MCDA Step Two: Analysis of Criteria using Criterium DecisionPlus

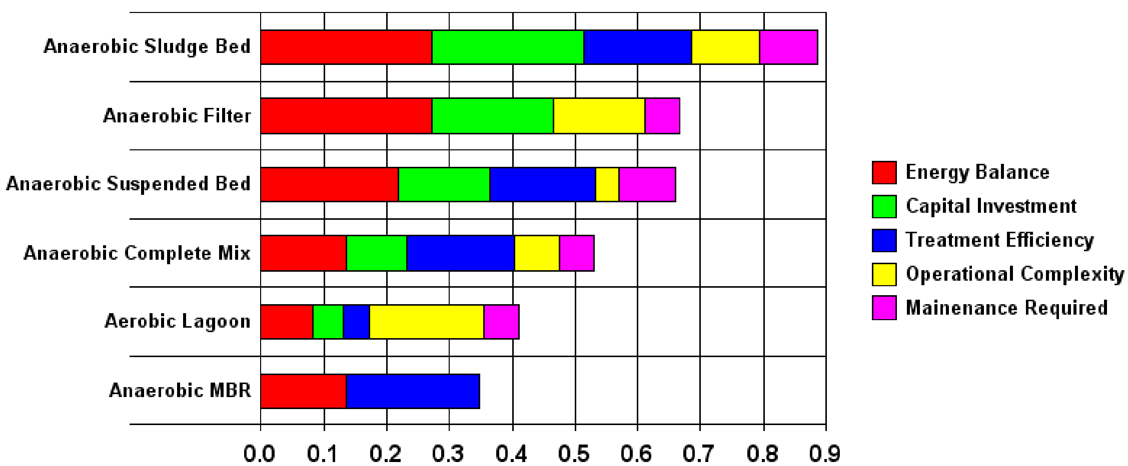
Reactor technology rankings were input into Criterium DecisionPlus and criteria importance weightings were applied accordingly (Table 3.3). Decision score results from Criterium DecisionPlus favored the anaerobic sludge blanket over other reactors for blackwater treatment application on the Foothills Campus (Figure 3.7).

Figure 3.7: Final decision score for each reactor



Examination of the breakdown of individual criterion ranking contributions to the final decision score for each reactor demonstrated the UASB did not have the highest score in each criterion. However, in consideration of the sum of all weighted criteria, the UASB outperformed all other reactors (Figure 3.8).

Figure 3.8: Contribution of each criterion ranking to final decision score



Since relative values of each weighted criteria ranking may not represent a value in the same order of magnitude, results from this analysis are potentially skewed.

3.4.3 MCDA Step Two: Analysis of Criteria using Ranking Comparison

An alternative method for comparison of reactor technology rankings and criteria importance weightings was used to select the most appropriate technology. The highest ranking reactor technology for each criterion was compared to criteria importance weightings (Table 3.6).

Table 3.6: Comparison of top ranked reactor technologies

Criteria	**Importance**	**Top Ranked Reactor Technologies**
Treatment Efficiency	7	Anaerobic MBR
Operational Complexity	6	Aerobic Lagoon
Maintenance Required	3	Suspended Bed & **Sludge Blanket**
Capital Investment	8	**Sludge Blanket**
Energy Input	9	**Sludge Blanket** & Anaerobic Filter
Energy Output	9	Suspended Bed, **Sludge Blanket**, Anaerobic Filter, Complete Mix, Anaerobic MBR

The sludge blanket (UASB) received a top ranking for four criteria. Three of the sludge blanket reactors top ratings were in criteria most important to CSU Department of Facilities Management, energy input, energy output, and capital investment. No other reactor technology received more than two top rankings. For this reason, the sludge blanket reactor design has been selected as the most appropriate technology for use in the CSU blackwater study. Design, operation, and results of the CSU blackwater study are presented in Chapter 4.

4.0 ANAEROBIC DIGESTION OF BLACKWATER STUDY

The objective of this study was to determine the potential for anaerobic digestion to serve as an ADWM blackwater treatment technology to treat blackwater from expansion of the Foothills Campus. Treatment efficiency was examined in a UASB reactor operated at a HRT between 2.6-4.0 days and OL) between 0.21-0.39 kg COD/m^3·d for a period of 108 days. Applicability of ADWM concepts were examined including resource recovery from blackwater, e.g. methane biogas production and nitrogen content in effluent for use as fertilizer. A fundamental comparison was performed between the energy balance of anaerobic and aerobic treatment.

4.1 Background

Blackwater is a highly concentrated, readily available, supply of organics and nutrients, which consists of toilet flushwater and flows from kitchens. Primary anaerobic treatment of blackwater produces methane biogas, a renewable source of energy, and provides nutrient enhanced effluent and solids which can be used as fertilizer and for soil enhancement. Blackwater contains the major portion of organics, nutrients, pathogens, hormones and other emerging contaminants, contained in household wastewater. Concentrating these risks into a reduced volume stream by separation of blackwater and graywater allows more effective control over contaminants of concern to public and human health, and facilitates more efficient, targeted treatment greatly reducing negative environmental impacts (Kujawa-Roeleveld et al. 2006). Further, graywater contains relatively low loads of contaminant risks compared to blackwater and, depending on application, can be readily reused after basic treatment. Drivers for source separation of blackwater and use of anaerobic digestion as a treatment method can be summarized as (modified from (Wendland 2009)):

- ***Safer sanitation:*** Contents hazardous to human and environmental health, pathogens, pharmaceutical residues and many other emerging contaminants are not spread into the water cycle. Introduction of health risks into surface waters can increase human contact with contaminants that pose severe hazard to public health, e.g. hormones.
- ***Renewable energy:*** Anaerobic digestion can provide a renewable source of methane biogas for cooking, lighting, heating, and electricity.
- ***Water savings:*** Due to the high solids handling ability of anaerobic digestion, use of pour or low-flush toilet technologies is encouraged, reducing the consumption of high quality drinking water.
- ***Organic fertilizer for agriculture:*** Because the liquid effluent from anaerobic digester contains high concentrations of nutrients and low numbers of pathogens, treated and disinfected digested blackwater can replace chemical fertilizer as a locally produced, readily available fertilizer.

4.1.1 Blackwater Characteristics

Blackwater characteristics can vary greatly depending on the variety of toilet used for delivery, and the amount of organic kitchen refuse incorporated into flows. Large variations in blackwater composition are expected, mainly due to the high particulate concentration of blackwater (Wendland et al. 2007). Blackwater from conventional flush toilets delivers considerably more diluted flow than blackwater from vacuum or low flush toilets. Several studies have characterized the composition of blackwater (Table 4.1) and blackwater with organic kitchen refuse (Table 4.2).

Table 4.1: Summary of literature blackwater (BW) characteristics (Wendland, 2009)

Literature source		(Kujawa-Roeleveld et al. 2006)	(Luostarinen 2005)	(Zeeman et al. 2007)	(Wendland 2009)
Parameter	**Unit**	**BW**	**Synthetic BW w/ primary sludge & toilet paper**	**BW**	**BW**
BW source		Vacuum Toilet	Synthetic	Vacuum Toilet	Vacuum Toilet
COD	mg/L	9,500 - 12,300	950	19,000	8,060 ± 2,950
dCOD	mg/L	1,400 - 2,800	120	5,000	2,440 ± 670
COD_{VFA}	mg/L	500 - 1,900	-	1,300	1,640 ± 470
$COD_{Particulate}$	mg/L	7,000 - 9,600	820	14,000	6,010 ± 2,790
TS	mg/L	-	670	-	6,530 ± 2,110
VS	mg/L	-	490	-	4,090 ± 1,830
TOC	mg/L	-	-	-	2,410 ± 720
NH_4-N	mg/L	600 - 1,000	4.5	1,400	1,111 ± 137
TN	mg/L	-	32	-----	1,495 ± 244
TP	mg/L	90 - 140	17	280	175
$COD_{Particulate}$:COD	-	76%	86%	74%	69%
COD/N/P	-	95/10/1	56/2/1	68/5/1	53/10/1
VS/N/P	-	-	-	-	24/10/1
E. coli	-	-	-	-	$9.1x10^7$

Table 4.2: Summary of literature blackwater (BW) and kitchen refuse (KR) characteristics

Literature source		(Wendland 2009)	(Wendland 2009)	(Kujawa-Roeleveld et al. 2006)	(Luostarinen & Rintala 2007)
Parameter	**Unit**	**KR**	**BW+KR**	**BW+KR**	**BW+KR**
KR:BW Ratio	-	-	1L BW:40g KR	-	30L BW:200g KR
BW Source	-	-	Vacuum Toilet	Vacuum Toilet	Synthetic
COD	mg/l	297,210	17,690 ± 4,530	13,300 - 22,900	2,268
dCOD	mg/l	80,330	6,780 ± 1,070	2,700 - 5,400	380
$COD_{Particulate}$	mg/l	216,880	10,260 ± 3,620	10,300 - 17,100	1,808
TS	mg/l	190,500	11,080 ± 3,040	-	-
VS	mg/l	172,370	7,920 ± 3,240	-	-
TOC	mg/l	80,690	5,420 ± 1,770	-	-
NH_4-N	mg/l	301	1,148 ± 111	600 - 1,300	6.4
TN	mg/l	4,901	1,503 ± 155		33
TP	mg/l	521	171	110 - 210	16
$COD_{Particulate}$:COD	-	73%	59%	-	-
COD/N/P	-	570/9/1	75/6/1	-	-
VS/N/P	-	330/9/1	46/6/1	-	-

Addition of organic kitchen refuse to blackwater can increase the influent loading considerably depending on local production and diet. Typically organic kitchen refuse is ground to prevent clogging and assist in hydrolysis of particulate matter in the reactor. Co-digestion of organic kitchen refuse with blackwater provides benefits over composting including increased methane production and ammonia content in effluent useable for fertilizer (Luostarinen & Rintala 2007). Anaerobic digester effluent is rich in soluble nutrients, a more readily available form for plant uptake (UNEP 2000).

4.1.2 Blackwater Treatment

Several studies have been performed recently showing successful anaerobic treatment of blackwater in UASB reactors at varying temperatures and organic loading rates (Table 4.3). Anaerobic biodegrability of blackwater has been reported at 81-85% at temperatures between 20-30°C (Elmitwalli et al. 2006). Typical operational OLRs for blackwater digesters have ranged between 0.33-0.96 kg COD/m^3·d. HRTs in many studies have been between 20-29 days, however biological treatment has produced removal of COD at 64% at a HRT of 0.5 days (van Voorthuizen et al. 2008). Blackwater digester operational temperatures have ranged between 15-37°C, however most studies remain in the mesophilic range. Generally, higher reductions in COD, VS, and pathogens have been observed at higher temperatures (Kujawa-Roeleveld 2005).

Studies monitoring UASB and complete mix reactors treating blackwater and blackwater with organic kitchen refuse have shown reductions in COD ranging between 61-82%. Operation of a UASB with membrane effluent filtration at 37°C observed a total of 91% COD reduction under high loading (2.28 kg COD/m^3·d) and low HRT (0.5 days) achieving 27% additional removal contributed to membrane filtration (van Voorthuizen et al. 2008). Addition of organic kitchen refuse slightly increased COD removal efficiency and increased VFA concentrations, indicating a

Table 4.3: Summary of literature high rate reactor performance

Literature source	Unit	(Kujawa-Roeleveld 2005)			(van Voorthuizen et al. 2008)	Wendland (2007)	Wendland (2007)
		UASBst1	**UASBst2a**	**UASBst2b**	**UASB w/ membrane**	**CSTR**	**CSTR**
BW Source	-	Vacuum Toilet	Vacuum Toilet	Vacuum Toilet	Low Flush Toilet	Vacuum Toilet	Vacuum Toilet
Substrate	-	BW	BW	BW & KR	BW	BW	BW & KR
Temperature	°C	15	25	25	37	37	37
HRT	days	29	29	27	0.5	20	20
OLR	kg COD/m^3·d	0.33	0.42	0.85	2.28	0.45	0.96
COD	mg/L	3,699 (61%)	2,733 (78%)	4,160 (82%)	104 (91%)	5,307 (61%)	13,632 (71%)
dCOD	mg/L	2,086 (-45%)	1,376 (31%)	1,718 (68%)	327 (38%)	-	-
VSS	mg/L	-	-	-	-	2,295 (51%)	5,720 (65%)
TN[a] / N_{kj}[b]	mg/L	960[b] (4.0%)	1,178[b] (16%)	1,289[b] (24%)	151[a] (14%)	(<2%)[a]	(<2%)[a]
NH_4-N	mg/L	826 (-16%)	1,068 (-6.8%)	1125	-	1,221 (-11%)	1,311 (-14%)
TP	mg/L	62 (40%)	62 (50%)	73 (65%)	-	-	-
E. Coli	CFU/100 mL	$1.11x10^{5}$ (87%)	$6.60x10^{3}$ (99.9%)	-	-	-	-
Fecal Coliforms	CFU/100 mL	$1.64x10^{9}$ (14%)	$1.15x10^{9}$ (49%)	-	-	-	-
Gas Production	Varies	6.4 L_{biogas}/day	8.3 L_{biogas}/day	19.5 L_{biogas}/day	104 L CH_4/kg $COD_{destroyed}$	393 L CH_4/kg $COD_{destroyed}$	380 L CH_4/kg $COD_{destroyed}$
Gas Content	%	-	72%	-	-	76%	65%

limiting methanogenic potential to utilize all of the increased VFA from organic kitchen refuse (Kujawa-Roeleveld 2005). COD in effluent has mainly been observed as dCOD and colloidal COD (Kujawa-Roeleveld 2005). Treatment of dCOD ranges between 45-68% in reported values. Effluent dCOD concentrations higher than influent have been contributed to a combination of escaping influent dCOD and hydrolysis of influent COD (Kujawa-Roeleveld 2005).

Higher NH_4-N values have been observed in reactor effluent, contributed to hydrolysis of urea and organically bound nitrogen which were more noticeable at higher temperature digestion (25°C vs. 15°C) (Kujawa-Roeleveld 2005). Precipitation of particulates has been observed as the main source of phosphorus removal in digestion of blackwater (Kujawa-Roeleveld et al. 2005). Phosphate precipitation was enhanced by addition of organic kitchen refuse (Kujawa-Roeleveld 2005). High ammonia concentrations in blackwater bring questions to the suitability of anaerobic digestion as a treatment process. However, Inhibition due to NH_4-N or VFA concentrations has not been noticed in anaerobic reactors treating blackwater (Elmitwalli et al. 2006) even at higher pH ranges (7.8) (Kujawa-Roeleveld 2005).

Effluent E. coli and fecal coliform concentrations in blackwater digesters without further treatment have not met standards for unrestricted irrigation set forth by the World Health Organization of 1000 CFU/100 mL (Kujawa-Roeleveld 2005) (Wendland 2009). An effluent E. coli concentration of 6.60×10^3 CFU/100 mL and fecal coliform concentration of 1.15×10^9 CFU/100 mL represent 99.9% and 49% reductions respectively in a reactor operated at 25°C (Kujawa-Roeleveld 2005).

Metals concentration in effluent from anaerobic blackwater reactors is an important consideration for application of effluent for irrigation purposes. A study measuring Cd, Cr, Cu, Mg, Ni, Pb, and Zn found metals concentrations in effluent to be meet water quality standards for irrigation, and come near metals concentration standards for commercial fertilizers, although the ratio of metals:phosphorus are exceeded on most measurements (Kujawa-Roeleveld 2005). This comparison makes a valuable argument for the safe reuse of effluent as a fertilizing source of irrigation water.

Energy balances performed on UASB reactors treating blackwater from vacuum toilets heated and unheated have shown positive energy balances (Kujawa-Roeleveld 2005). As expected, lower temperatures reduced methanogenic activity in blackwater digesters, as biogas production rates of 6.4 L/d and 8.31 L/d were observed UASB reactors operated at 15-25°C respectively (Kujawa-Roeleveld et al. 2005). Biogas production observed values between 6.4-8.3 L/day which increased to 19.5 L/day after addition of organic kitchen refuse without increasing HRT substantially (Kujawa-Roeleveld 2005). Also, potential to produce between 104-393 L CH_4/kg $COD_{destroyed}$ has been observed (Wendland et al. 2007) (van Voorthuizen et al. 2008).

Inoculation reduced timing for stabilization of particulate removal, in a comparison of two reactors, one inoculated and one not inoculated, more rapid stabilization of particulate removal was noticed in the inoculated reactor (Kujawa-Roeleveld et al. 2005). A sum layer was noticed for some time during initial operation in reactors operated at temperatures of 15-25°C and OLRs of 0.33-0.42 kg COD/m^3·d respectively, which disappeared after reactor operation stabilized (Kujawa-Roeleveld 2005). Digestion of blackwater provides generally stable reactor pH and typically adjustment is not necessary due to the buffering capacity of blackwater (Wendland 2009).

4.1.3 Full Scale Blackwater Treatment Operations

Europe, India, and China have shown dedication to sustainable and responsible management of wastewater through implementation of full scale projects incorporating anaerobic digestion as a wastewater treatment technology for concentrated wastewaters.

4.1.3.1 Lübeck-Flintenbreite, Germany (OtterWasser GmbH 2003)

The Flintenbreite housing development in Lübeck, Germany successfully incorporated sustainable wastewater management into initial design. Blackwater is diverted to a central holding tank using water reducing vacuum toilets and organic kitchen refuse is combined with blackwater for digestion. Methane biogas produced in the digester is used to supplement natural gas for combined heat and power generation. Sludge from anaerobic digestion is used for agricultural application.

4.1.3.2 Sneek, The Netherlands (Wendland 2009)

A blackwater collection and treatment system began operation in 2006, serving 32 rental units in Sneek, The Netherlands. Vacuum toilets are used to transport blackwater to two 6m^3 UASB septic tanks for treatment and biogas recovery. Post treatment of effluent recovers residual COD and nutrients for reuse. Although the current scale of the operation is not feasible, size will be up scaled in years to come after feasibility of the post treatment is determined.

4.1.3.3 China and India

Programs to promote biogas digesters in China and India began in the 1970's and 1980's when fossil fuels became increasingly expensive. A Chinese campaign in the 1970's developed a large number of anaerobic digesters throughout the country, however many of them failed in following years due to lack of maintenance and upkeep. Today, there are greater than 5 million anaerobic digesters in China between 6-10 m^3 serving individual households which are fed with organic wastes such as animal and human excreta and organic kitchen refuse (Wendland 2009). Historically, biogas production in China focused on small scale biogas production plants. However, there is a large move to substantially increase the amount of biogas produced in China over the course of the next few decades through construction of large scale digesters to improve the renewable energy supply of the nation (Anon 2009).

A ‘National Biogas and Manure Management Programme’ (NBMMP) in India has promoted development of small scale biogas plants in rural areas since the early 1980’s (Anon 2010). More recently, nearly 100 large scale biogas plants are in operation treating blackwater from public pour flush toilet systems (Wendland 2009).

4.1.4 Study Objectives

The objective of the demonstration portion of this study was to examine potential for anaerobic digestion to serve as an effective and sustainable blackwater treatment technology for treatment of Foothills Campus blackwater. Much of the existing literature on anaerobic digestion of blackwater comes from laboratory scale studies and full scale implementations from model housing developments in the European region. The main technologies used in these studies are UASB septic tanks (similar to a standard UASB reactor), CSTR reactors, and accumulation tanks. Results show great potential for production of energy from blackwater as well as reuse of treated effluent as a fertilizing source of irrigation water. Further study is needed on application of effluents from digesters for irrigation purposes. Studies on metals and VFA concentrations in effluent and their potential effect on plants would provide a basis for legislation regarding this effluent use. Study is also needed on the reduction of pathogens by digesters treating blackwater. Incorporation of some type of low maintenance disinfection such as UV light should be considered. A full scale study incorporating reliable capture and use of methane produced by an anaerobic digester treating blackwater would show the potential for the positive energy balance which anaerobic digestion has potential to offer. Also, research on the fate of emerging contaminants including hormones and pharmaceuticals in blackwater digestion systems (including sludge and effluent) would help regulators gauge the safety of reuse.

The results from this study are expected to provide insight into the technical feasibility of anaerobic treatment of blackwater from conventional flush toilets in a research campus setting. The majority of studies performed previously have used blackwater originating in residential developments using low flush or vacuum toilet technologies which provide greatly reduced volumes and increased concentrations of blackwater. Results contribute to the knowledge base of anaerobic digestion of blackwater and the potential benefits and risks posed by this concept. Treatment efficiency, nutrient concentrations, biogas production, and operational issues are explored and discussed.

4.2 Materials and Methods

The source of blackwater being studied was a research and office building located on the Foothills Campus. Plumbing for the building is separated into three streams: blackwater, labwater, and water from hand sinks. Toilets, emergency showers, kitchen sinks, and floor drains contribute to the blackwater line, although toilets supply the majority of flow. Toilets and urinals used approximately 5 L and 1.5 L per flush respectively. Around 285 L of blackwater was produced daily during weekdays.

4.2.1 Experimental Setup

4.2.1.1 Batch Assays

A series of initial testing was carried out on blackwater sampled from the CSU Atmospheric Science Chemistry building (CSU blackwater) to determine the treatability of the substrate. Four rounds of initial tests were performed. Earliest testing involved determination of methane production potential through biochemical methane production (BCMP) tests. BCMP tests provided somewhat lower biogas methane concentration measurements than comparable literature which prompted further examination of blackwater. Aerobic and anaerobic biodegradation tests were performed to ensure sufficient destruction of COD was achievable.

pH stability tests were performed to ensure pH levels were not causing inhibition or toxicity of methanogens. Cleaning production toxicity testing was performed to test for toxic or inhibitory effects of expected concentrations of cleaning products used in toilets and on floors on seed organisms.

BCMP tests were performed as 200 mL batch anaerobic assays modeled after the bioassay method for testing biodegradation outlined in (Owen et al. 1979). Three blackwater assays were prepared containing 180 mL of inoculated nutrient solution (Owen et al. 1979) and 20 mL of CSU blackwater substrate. Three controls were also prepared containing 200 mL of inoculated nutrient solution to deduct methane production contributed from the inoculated nutrient solution. Assays were stored on an incubated shaking table at 35°C. Gas samples were analyzed for methane concentration six times over a 42 day period. Measurements were taken more frequently during the beginning of the test when gas production was highest. Gas volume extracted during each sampling was also recorded by inserting a 20 mL syringe through each assays rubber stopper (Owen et al. 1979). Methane volume produced for each sampling was calculated using measured volumes and methane percentages in biogas measured by gas chromatograph (GC).

Aerobic and anaerobic biodegradation tests were configured as simple batch reactors consisting of inoculum, nutrient solution (Owen et al. 1979) and blackwater substrate for a combined total assay of 1000 mL. Aerobic and anaerobic waste sludge from the Drake Municipal Water Reclamation Facility in Fort Collins, Colorado was used as inoculum in aerobic and anaerobic tests respectively. A total of six aerobic and six anaerobic tests were run, three containing blackwater substrate and three controls for each type. Aerobic and anaerobic biodegradation assays contained 500 ml blackwater, and 500 ml aerobic or anaerobic defined media (inoculum & nutrient solution). Aerobic and anaerobic controls contained 500 mL DI water and 500 mL defined media nutrient solution. Aerobic assays were stored at ambient room temperature and completely mixed throughout the experimental period. Anaerobic assays were stored in a shaking incubator at 35°C for 23 days. DOC and dTN were measured five times over a period of 23 days. For aerobic tests, 15 ml samples were taken from assays by removing foam the stopper and pouring composite sample into a 15 ml sample vial. Anaerobic tests were sampled by inserting a 16 gauge syringe through the rubber stopper located in the mouth of the test bottle and extracting 15 mL of sample. Extracted sample was then placed in a 15 ml sample vial for transport. Prior to analysis, samples were filtered through 0.45 µm filter cartridges. Sample analysis was performed the day of sampling.

pH stability tests consisted of three 200 mL assays containing 100 mL of inoculated nutrient solution (Owen et al. 1979) and 100 mL of CSU blackwater substrate each. Assays were prepared in an anaerobic chamber and stored on a shaking incubator at 35°C over a period of 16 days. pH was measured nine times inside of an anaerobic chamber by removing the vial stopper and inserting a pH electrode into each assay. During five of the pH measurements, samples were drawn to measure dNH_3-N, and dCOD.

Cleaning product toxicity was tested in four 200 mL assays. Assays were run in triplicate containing the following:

1. Toilet bowl cleaner: 0.3 mL toilet bowl cleaner, 0.2 g glucose, 20 mL anaerobic inocula, 180 mL DI water
2. Toilet bowl and floor cleaner: 0.3 mL toilet bowl cleaner, 0.3 mL floor cleaner, 0.2 g glucose, 20 mL anaerobic inocula, 180 mL DI water
3. Glucose: 0.2 g glucose, 20 mL anaerobic inocula, 180 mL DI water
4. Control: 20 mL anaerobic inocula, 180 mL DI water

The volume of toilet bowl cleaner used for experiments was calculated assuming the recommended amount of cleaner is applied to four 5 L flush toilets and two 1.5 L flush urinals and is collected and diluted in a 114 L compositing tank. The same amount of floor cleaner was used as an over approximation of the actual amount that would get flushed down floor drains and into the same 114 L compositing tank. In addition to the three previously mentioned assays, a 200 mL blackwater assay was run in triplicate to test uninhibited biodegradation. The blackwater assay contained 100mL blackwater and 100 mL inoculated nutrient solution (Owen et al. 1979). Because no separate control was run, COD measured from the cleaning product toxicity tests was divided in half to account for the dilution of blackwater assays and used as a control value. Seed for all assays was taken from a UASB anaerobic reactor treating brewery waste at the New Belgium Brewery in Fort Collins, Colorado. Cleaning product assays were sampled three times over a period of 8 days for dCOD and the mean of triplicate results was used for analysis. Blackwater assays were measured for COD and dCOD during this period.

During blackwater sampling for batch assays and initial characterization, efforts were made to capture composite samples. One 15.2 cm PVC blackwater sewer line leaves the study building and spans across the bottom of a concrete vault adjacent to the building. A valve was installed on this line and a screw cap access was placed upstream of the valve. Sampling was performed by temporarily shutting the valve and routing blackwater through the screw cap access into a 114 L drum (Figure 4.1). Once filled, the 114 L drum was disconnected from the blackwater line. Wastewater within the drum was composited by placing a laboratory bench mixer fitted with a 122 cm metal rod with two propellers into one of two access ports at the top of the drum. One propeller was oriented at the bottom of the drum to kick up settled solids and the second propeller was oriented midway in the barrel to assist in mixing. While the mixer was operating, a hand operated 2.5 cm drum pump was inserted into the second drum access port and approximately 12 L of sample was withdrawn and taken to the laboratory for analysis. All BCMP testing and initial sample characterization experiments were performed the same day of sampling.

Figure 4.1: Compositing device used during initial blackwater sampling

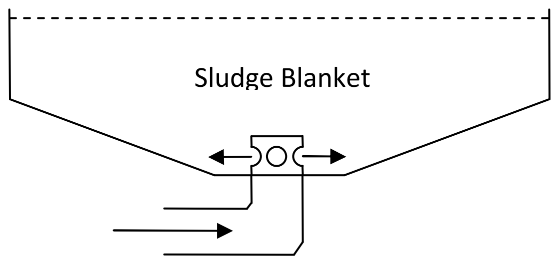
4.2.1.2 Anaerobic UASB

A demonstration scale 108 L UASB reactor was constructed at the Atmospheric Science Chemistry Building on the Foothills Campus of CSU. Guidance for design of a UASB reactor for treatment of blackwater was taken from literature available on UASB design criteria (Lettinga et al. 1991) (Souza 1986) (Vieira & Garcia 1992). Literature was not available on the design of UASB for blackwater digestion. However, information from previous studies using the UASB design for blackwater digestion was incorporated including OLRs, HRTs, and width to height ratios (Kujawa-Roeleveld et al. 2005) (Luostarinen et al. 2007).

The UASB was constructed using a 114 L conical bottom high density polyethylene tank manufactured by Ronco Inc. A 10 cm wide acrylic window was installed vertically along the cylindrical portion of the tank to enable examination of the sludge blanket and reactor volume. The reactor was approximately 476 mm in diameter and 806 mm tall. The conical portion was

63.5 mm in height. The liquid volume of the reactor was approximately 108 L and head space consisted of the remaining volume. Blackwater was fed into the sludge blanket through the bottom of the reactor. The feed inlet consisted of a 2.5 cm PVC schedule 40 pipe cap at the bottom of the reactor (Figure 4.2). This cap was screened with four 1.9 cm diameter holes situated to allow flow to initially distribute horizontally through the sludge blanket.

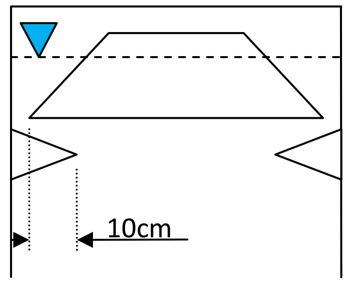
Figure 4.2: UASB influent feed design



An inlet spacing of one inlet per 1-2 m^2 and 7-10 m^2 is recommended for diluted and concentrated feedstocks respectively (Souza 1986). The spacing achieved in the demonstration reactor was one inlet per 0.17 m^2. Due to the size of the reactor, it was not possible to achieve a higher spacing. An inlet height of 20 cm from the tank bottom to the inlet is recommended (Vieira & Garcia 1992). This spacing is for sludge accumulation and removal. Due to the short run time of the reactor (108 days), sludge production was expected to be minimal; thus, this spacing was not taken into account.

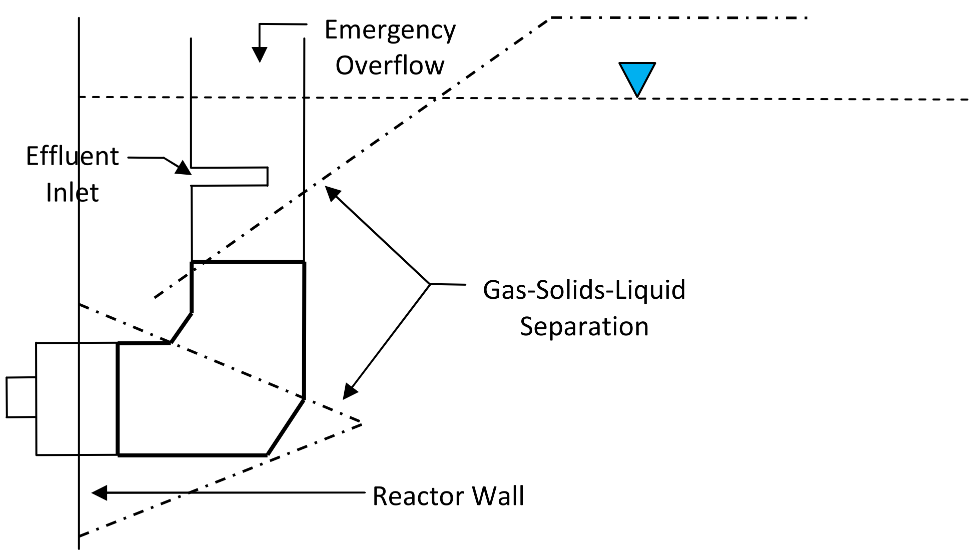
A frustum cone settler and circular wedge deflector were used for gas-solids-liquid separation in the UASB reactor (Figure 4.3). The frustum cone settler had approximate side slope of 60° and contact surface area of 0.14 m^2. The circular wedge deflector had an approximate slope of 25° on the top and bottom surfaces. Overlap between the cone settler and wedge deflector was approximately 10 cm. A settler slope between 45-60° is recommended for effective solids settling within the reactor (Souza 1986) (Lettinga et al. 1991). A deflector/settler overlap of at least 10 cm is advised (Souza 1986) (Lettinga et al. 1991).

Figure 4.3: UASB gas solids liquid separator design



Little information was available for design of an effluent weir for demonstration scale reactors. The main focus of design was to limit the introduction of floating solids, fats, and oils from entering the effluent stream. Final design was based off the single effluent outlet similar to that shown in a similar study (La Motta et al. 2008). The effluent weir had a roughly 6.4 mm tall by 76 mm long intake slot approximately 5 cm below the FWS (Figure 4.4). Around 5 cm above the FWS, the effluent pipe was open to allow emergency overflow out of the reactor in case of weir clogging. The effluent piping was 5.1 cm schedule 40 PVC inside of the reactor, reduced to 13 mm schedule 40 PVC outside of the reactor.

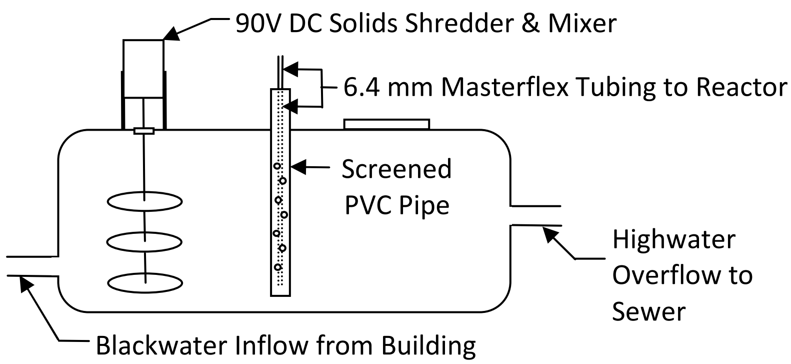
Figure 4.4: UASB effluent weir design



Heating of the reactor was performed internally using a Finnex 300 W titanium aquarium heater with a HC810 digital controller. The heating coil was mounted horizontally in the center of the tank and the thermostat was placed at the top of the tank to ensure temperature uniformity throughout the reactor.

A compositing tank was constructed out of a horizontally oriented 114 L tank. Blackwater from the building plumbing was gravity fed into the compositing tank through a 51 mm diameter PVC pipe. A solids shredder & mixer were constructed out of 178 mm circular saw blades (Figure 4.5).

Figure 4.5: Blackwater compositing system



Blackwater was fed to the reactor through 6.4 mm Masterflex tubing. A peristaltic pump was used to draw blackwater through this tubing. Masterflex tubing was enshrouded within the compositing tank by a perforated 3.2 cm diameter PVC pipe to prevent clogging between the compositing tank intake and reactor inlet. A 5.1 cm PVC overflow pipe routed blackwater to the sewer as fresh blackwater entered through the inflow pipe. Blackwater flow from the building was in the range of 285 liters per day during weekdays. All blackwater was routed through the compositing tank, providing a retention time of approximately 8.5 hours Monday-Friday. During weekends flow was extremely variable depending on building use and retention times in the compositing tank varied accordingly. Reactor effluent was sent directly into the municipal sewer (Figure 4.6, Figure 4.7).

Figure 4.6: UASB reactor system

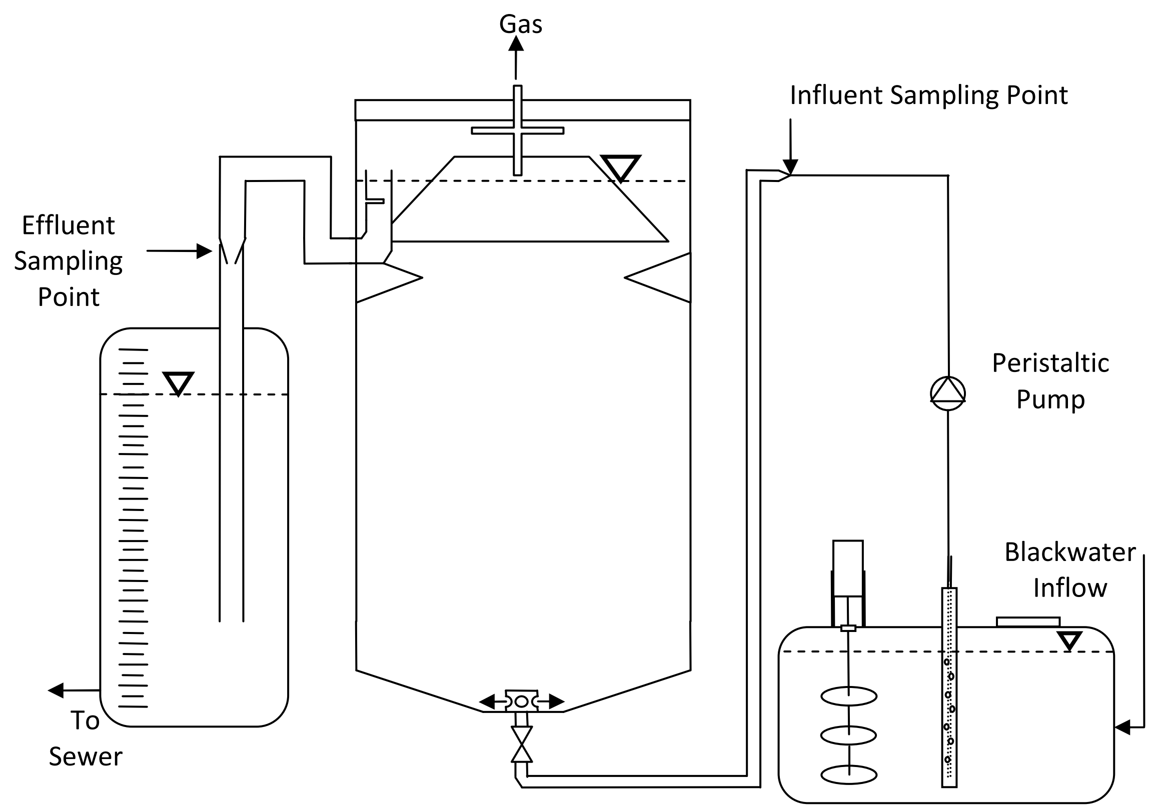


Figure 4.7: Reactor, compositing tank, and equipment configuration

4.2.2 Reactor Operation

Initially the reactor was filled with potable water. Potable water was displaced with blackwater over a period of one week through influent pumping. At this time the reactor was inoculated with 26.5 L of sludge from the New Belgium Brewery UASB reactor treating primary wastewater from brewing processes. Temperature was set at 34°C on the temperature control during startup. The reactor was operated at varying HRT and OLR over a period of 108 days. Startup of the UASB reactor began on September 1, 2009 with the inoculation of the reactor. Maximum organic loading rate during startup was recommended at 0.5 kg COD/m^3·d (Souza 1986). To encourage formation of sludge granules efficient at high feed rates, the reactor was initially operated at a mean OLR of 1.18 kg COD/m^3·d for a period of 20 days (Table 4.4).

Following startup, reactor performance was monitored continuously for two distinct periods, identified as Phase I (31 day duration) and Phase II (14 day duration). Reactor inflow was shut off between days 83-88 due to extenuating circumstances. Two operational stages can be identified during Phase I when the reactor was operated at a mean HRT of 4.0 days between day 49-62 (Phase Ia) and HRT of 2.6 days between day 63-80 (Phase Ib) (Table 4.4). During Phase II, the reactor HRT varied between 2.6 and 4.0 days with a mean HRT of 3.7 days.

Table 4.4: Summary of mean reactor operation parameters by phase

	Days	**Flowrate**		**HRT**		**OLR**	
		(mL/min)		(days)		(kg COD/m^3·d)	
Startup	0 - 20	98	-	0.8	-	1.18	-
	21 - 48	24	-	3.1	-	0.29	-
Phase I	49 - 62	19	(2.8)	4.0	(0.5)	0.21	(0.03)
	63 - 80	29	(4.6)	2.6	(0.5)	0.39	(0.06)
Phase II	94 - 108	22	(4.8)	3.7	(1.4)	0.27	(0.13)

*Standard deviations are shown in parenthesis

4.2.3 Reactor Sampling

Instantaneous draw samples of the reactor influent and effluent were taken for analysis at influent and effluent sampling locations (Figure 4.6). During reactor monitoring Phase I and II, samples were drawn three times per week on average. Influent and effluent samples were taken in 250 mL and 500 mL vials respectively and taken to the laboratory for analysis. Only COD and dCOD were measured during the startup phase to gauge reactor performance. In Phase I and Phase II COD and dCOD were measured for each sampling event. The number of experimental data points available for calculation of parameters varies, as some data points had to be eliminated for reasons including inconsistencies in results and experimental error (Table 4.5).

Table 4.5: Number of experimental data points available for calculations by parameter and phase

	Startup[2]		**Phase I**		**Phase II**	
	Influent	*Effluent*	*Influent*	*Effluent*	*Influent*	*Effluent*
COD	3	3	14	14	7	7
dCOD	3	3	14	14	7	7
Biogas CH_4 %	-		8		4	
Biogas Volume	-		-		1	
dNH_3-N	-	-	10	10	6	6
TSS	-	-	12	12	7	7
VSS	-	-	11	11	7	7
DOC[1]	-	-	11	11	5	5
TN[1]	-	-	11	11	5	6
E. coli	-	-	-	-	4	5
Fecal Coliforms	-	-	-	-	4	4
Flowrate	-	-	13	-	7	6
pH	-	-	12	12	7	7
Temperature	-	-	11	11	7	7

[1] DOC, TN, and CH_4 measured until day 80

[2] Flowrate maintained during this period

Flowrate was measured at both influent and effluent sampling points during each sampling. Inflow was measured using a graduated cylinder and stopwatch. Adjustments were made as necessary to the peristaltic pump to match the required flowrate. A graduated tank was used for effluent flow measurement during Phase II. Effluent was routed into the tank and the total volume of effluent produced was recorded over sampling intervals. Mean COD influent concentration over the study period was used to calculate OLR during startup. COD measured during each sampling event was used to calculate OLR for periods between sampling events.

4.2.4 Analytical Methods

4.2.4.1 Liquid and Solids Analysis

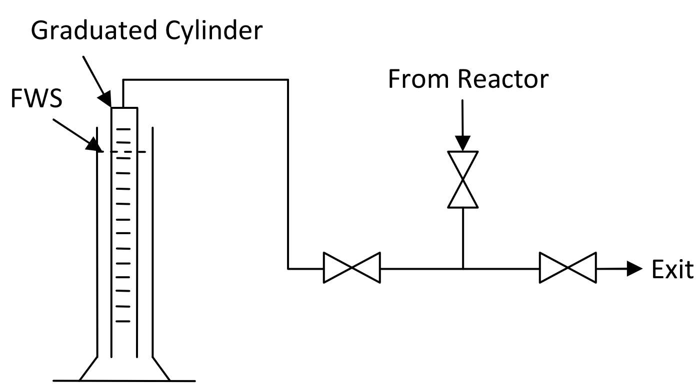
All samples subject to dissolved measurements were filtered using 0.45μm syringe filters. COD and dCOD testing was performed using a closed reflux colorimetric method in accordance with standard method SM 5220 D (Clesceri et al. 1998). DOC was measured on a Shimadzu TOC-V combustion analyzer conforming to standard method SM 5310 B (Clesceri et al. 1998). dTC minus dIC was used for determination of DOC. Standards were prepared for 0, 100, and 200 mg/L for both dTC and dIC. dTN was measured on a Shimadzu TOC-V combustion analyzer with a TNM-1 TN chemiluminescence detector module. Standards were prepared for 0, 100, and 200 mg-dTN/L. To measure dNH_3-N, pH of samples was adjusted above 11 to ensure all dNH_4-N was in the form of dNH_3-N. dNH_3-N was measured using an Accument gas sensing electrode by Fisher Scientific conforming to standard method SM 4500-NH_3 D (Clesceri et al. 1998). Standards were prepared for 10, 100, and 1000 mg-dNH_3-N/L. All ammonia nitrogen results are provided as dNH_3-N. TSS were measured by filtering a chosen volume of sample on Whatman 934-AH glass fiber filters and evaporating liquid at 105°C for a minimum of one hour, conforming to standard method SM 2540 D (Clesceri et al. 1998). Filters were cooled in a desiccator, weighed, and then used for VSS analysis. VSS were measured by firing filters from TSS analysis at 550°C for at least 30 minutes, conforming to standard method SM 2540 E (Clesceri et al. 1998). Filters were then cooled in a desiccator and weighed. IDEXX Colilert

Quanti-Tray 2000 tests were used to perform E. coli and fecal coliform measurement in accordance with standard method SM 9223 B (Clesceri et al. 1998). It was necessary to dilute influent by a factor of 1:1,000 and effluent by a factor of 1:10,000 to achieve measurable colony counts. A YSI ProfessionalPlus Multiparameter instrument was used to measure temperature and pH during each sampling event. Measurement was performed in 100 mL vial grab samples at the reactor site.

4.2.4.2 Gas Analysis

Biogas volume was measured throughout the 12 week experimentation. However, problems with collection systems were encountered. Initially, leaks in the reactor and tubing for gas collection caused inaccurate measurement in the wholly imprecise measurement device in place. Once leaks were sealed, a larger collection device was necessary. Effective collection and measurement were difficult with the constructed system. After the first successful gas collection period in the new enlarged device, tubing from the reactor to the collection device was pinched. It is believed that this pinch led to breach of the reactor seal, as gas built up and was forced to find an exit. A second method was examined which measured flow over a short period at the end of testing using liquid displacement in a graduated cylinder (Figure 4.8). Five, two hour gas production measurements were taken over a three day period using this method.

Figure 4.8: Graduated cylinder short term gas measurement device



Biogas was collected in 1 L tedlar gas bags and analyzed using a HP5980II GC incorporating a Hayesep Q, 80/100 mesh packed column and thermal conductivity detector. A temperature of 100°C was used for the injector, detector, and oven. This temperature was modified slightly from the recommended chromatogram to work effectively with the HP GC. Standards of 20%, 40%, 60%, and 80% CH_4 were made by diluting 5µL/15µL, 10µL/10µL, 15µL/5µL, 20µL/0µL respectively in a 50µL syringe of 80% CH_4/ N_2 gas contained in 1 L tedlar bags.

4.3 Results

4.3.1 Blackwater Quality

Initial quality parameter averages for blackwater from the CSU Atmospheric Science Chemistry building were generally higher than averaged experimental results from reactor influent blackwater (Table 4.6).

Table 4.6: Mean blackwater characteristics from initial sampling and reactor influent analysis

	Units	# Samples	Initial Blackwater[2]		# Samples	Reactor Influent Blackwater[2]	
TSS	mg/L	3	1,883	(354)	19	435	(147)
VSS	mg/L	3	1,721	(333)	18	379	(119)
COD	mg/L	3	1,983	(227)	24	932	(244)
dCOD	mg/L	4	671	(160)	24	234	(47)
BOD_5[1]	mg/L	-----	1,686		-----	793	
DOC	mg/L	5	127	(7)	16	53	(15)
dTN	mg/L	4	145	(24)	11	77	(11)
dNH_3-N	mg/L	2	127		12	102	(8)
pH	-----	-----	-----		19	8.9	(0)
E.coli	CFU/100 mL	-----	-----		4	3.6×10^6	
Fecal coliforms	CFU/100 mL	-----	-----		4	3.8×10^6	

[1] Calculated from measured BOD:COD of 0.85
[2] Standard deviation shown in parenthesis

The discrepancy in blackwater characterization results can be contributed to three major influences on the reactor influent blackwater: increased hydrolysis rates in the compositing tank from mixing, partial degradation of the substrate in the compositing tank during the retention time experienced (up to 8.5 hours on weekdays and greater on weekends) and restriction of solids entry due to use of 6.4 mm peristaltic tubing for influent pumping.

Particulate COD (>0.45 µm) in the initial blackwater represents 66% of total COD, whereas particulate COD in the reactor influent represents 75% of total COD. The VSS:TSS ratio for initial blackwater was 91% and for reactor influent was 87%. An increased particulate COD fraction and decreased volatile suspended solids fraction in reactor influent show that readily available substrates are being partially degraded in the compositing tank. High pH levels in reactor influent are contributed to the hydrolysis of organic urea nitrogen to ammonia in the compositing tank. High concentrations of indicator E. coli and fecal coliform colony forming unit counts are present in the influent blackwater as expected.

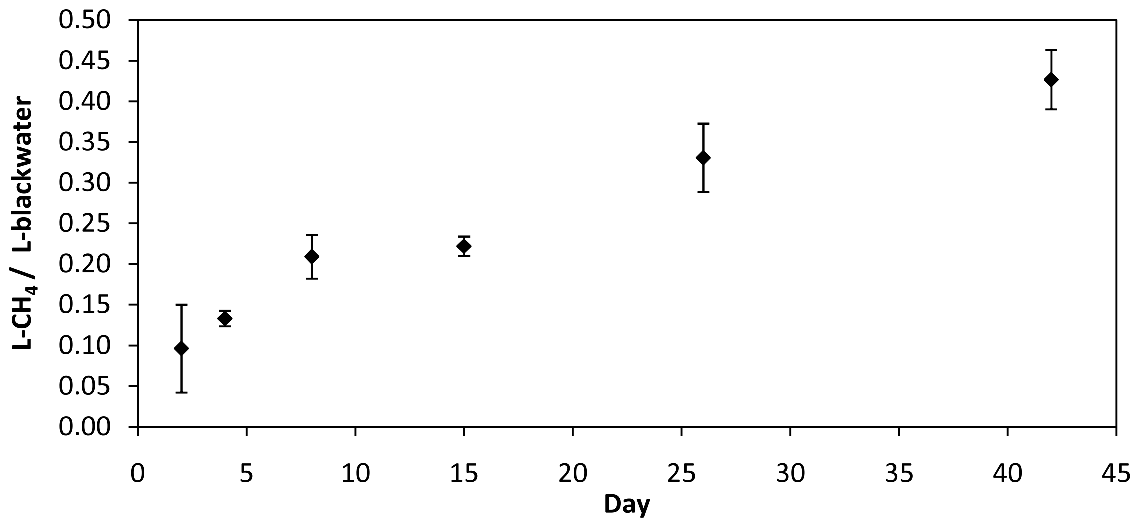
The ratio of dNH_3-N:dTN in initial blackwater (88%) was much higher than that measured for reactor influent blackwater (71%). Some portion of the ammonia may have been converted to nitrite. However dTN in reactor influent (77 mg/L) was much lower than that measured in initial blackwater (145 mg/L). For this reason and the elevated rate of corrosion noticed within the reactor vault, it is likely that a fraction of the nitrogen escaped the compositing tank as ammonia gas.

4.3.2 Batch Assays

4.3.2.1 Biochemical Methane Production Potential (BCMP)

Final cumulative methane production from BCMP testing provided 0.43 L CH_4/L blackwater when tests were performed at day 42 (Figure 4.9). Using the mean initial blackwater characterization COD value (1,983 mg/L), methane production based on COD can be calculated as 217 L CH_4/kg COD_{input}. This value compares to a similar batch study performed using blackwater from vacuum toilets, which showed 209 L CH_4/kg COD_{input} (Wendland 2009). While

Figure 4.9: Cumulative CH_4 production from CSU blackwater



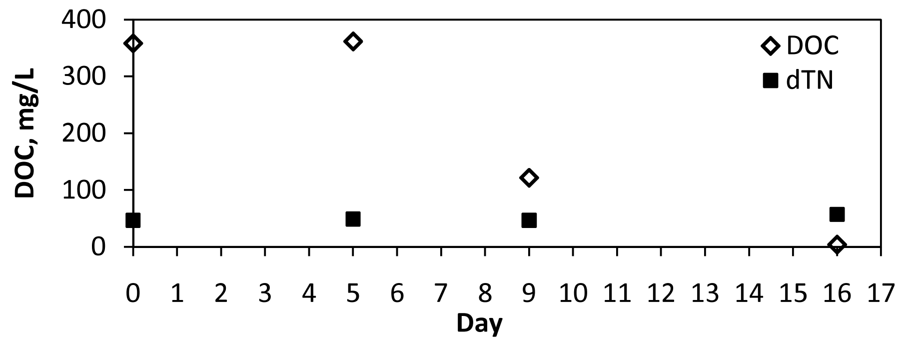
volume calculations show similar results to previous study, biogas methane concentration was comparatively low, between 11-27%. Concerns of methanogenic inhibition and toxicity necessitated further testing.

4.3.2.2 Aerobic & Anaerobic Biodegradation

Biodegradation testing was performed on blackwater samples to determine aerobic and anaerobic treatment potential. Similar trends were experienced in dTN for both assays over the first nine days (Figures 4.10 & 4.11). On the second sampling date, 5.0% and 6.3% dTN increases were experienced in anaerobic and aerobic assays respectively. dTN is the sum of all dissolved species of nitrogen including nitrite, nitrate, organic nitrogen and ammonia. Nitrogen speciation was not performed. However, it is possible that the increase was caused by nitrogen fixing bacteria releasing nitrogen into solution during the initial growth phase. An increase in dTN was also experienced at the end of testing in anaerobic tests which may be contributed to soluble microbial product release via endogenous decay.

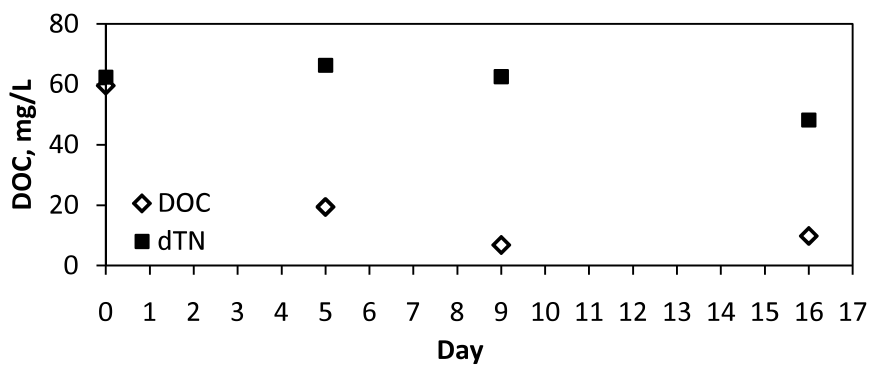
DOC concentration did not change substantially between the first two sampling periods in anaerobic tests. Anaerobic assays were not prepared in an anaerobic environment and therefore this trend could represent a biomass startup phase. Effective DOC reduction was experienced between days 5 and 16. A DOC value of 3.65 mg/L was measured during the last sampling date in anaerobic assays. This value is unexpectedly low and is contributed to experimental error.

Figure 4.10: Anaerobic biodegradation results



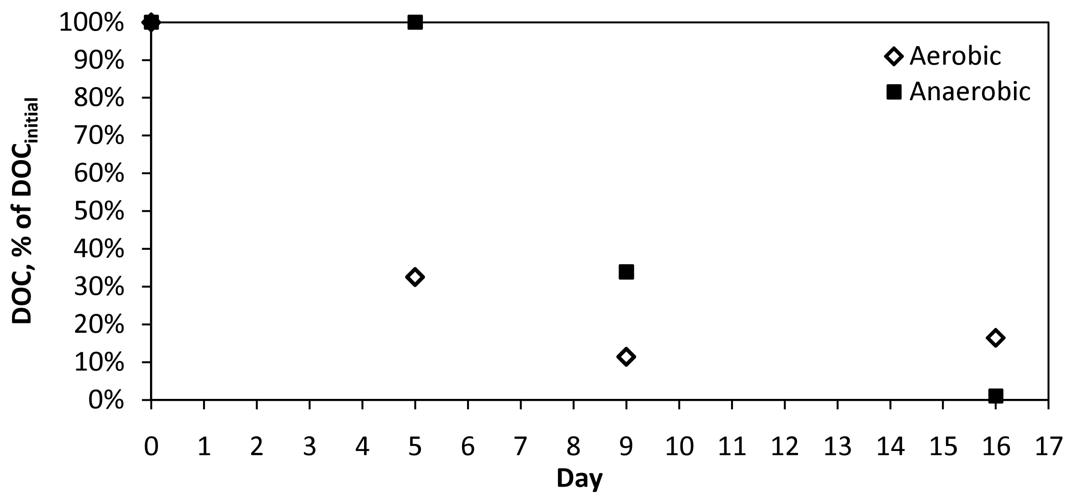
In the Aerobic biodegradation tests, a slight increase in DOC was observed at the end of testing (Figure 4.11), likely a result of endogenous decay. Decreasing dTN in the final days of testing is noticed which may be contributed to biological uptake of nitrogen.

Figure 4.11: Aerobic biodegradation results



Results showed 89% and 66% reduction in blackwater DOC at day 9 in aerobic and anaerobic assays respectively (Figure 4.12). Day 9 was chosen for comparison because of the experimental error which occurred in anaerobic tests during day 16.

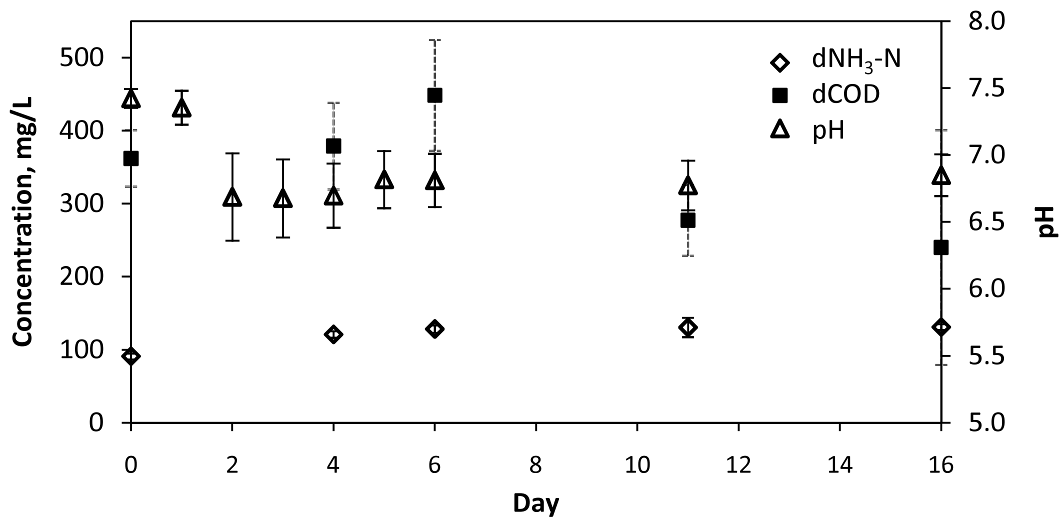
Figure 4.12: DOC degradation during experimentation



As expected, substrates were degraded at a higher rate in an aerobic environment compared to an anaerobic environment. It is anticipated that anaerobic tests had some initial startup period due to residual oxygen present in the headspace of each assay. For this reason, it can be expected anaerobic processes would degrade waste more rapidly in an acclimated reactor than what the results of these tests provide. Biodegradation testing results showed effective DOC reduction was occurring in assays even while methane production rates were lower than expected. To determine potential causes for methane inhibition another set of batch assays were conducted. pH was monitored in assay tests to ensure acid production was not causing inhibition or toxicity of methanogenic organisms. Results are averaged between all three assays. Initial pH measured 7.4 and stabilized near 6.8 at the end of experiments (Figure 4.13). This range is well within generally recommended pH of 6.5-7.5 for anaerobic digestion.

dNH_3-N ranged between 91-131 mg/L during the experimental period. NH_4-N inhibition is observed as low as 1,500 mg/L, well above measured values (Tchobanoglous et al. 2003). NH_3-N inhibition is observed at 150 mg/L (Kroeker et al. 1979). Measured mean initial blackwater dNH_3-N concentration (127 mg dNH_3-N/L) was less than the minimum level for inhibition. Therefore, dNH_3-N was not at inhibiting levels.

Figure 4.13: pH stability test results

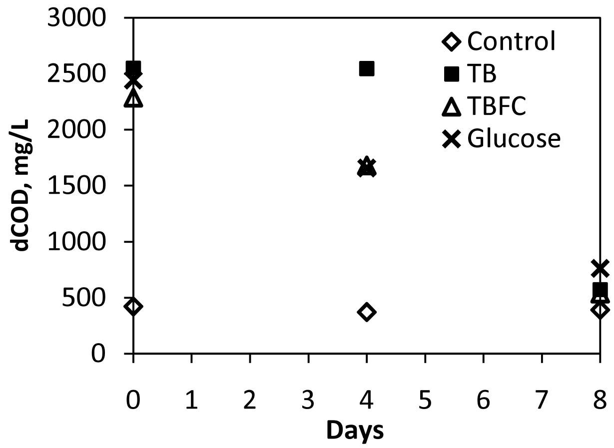


Controls containing nutrient solution were not run during experiments and therefore the contribution of measured parameters from blackwater substrate and inoculated nutrient solution cannot be extracted. COD and dCOD were reduced by 51% and 34% respectively. However because controls were not run, these values represent reduction of total COD and dCOD for entire assay and not just the blackwater fraction. Results show that methanogenic inhibition is not being caused by pH or nitrogen concentrations within assays.

4.3.2.3 Cleaning Product Toxicity Testing

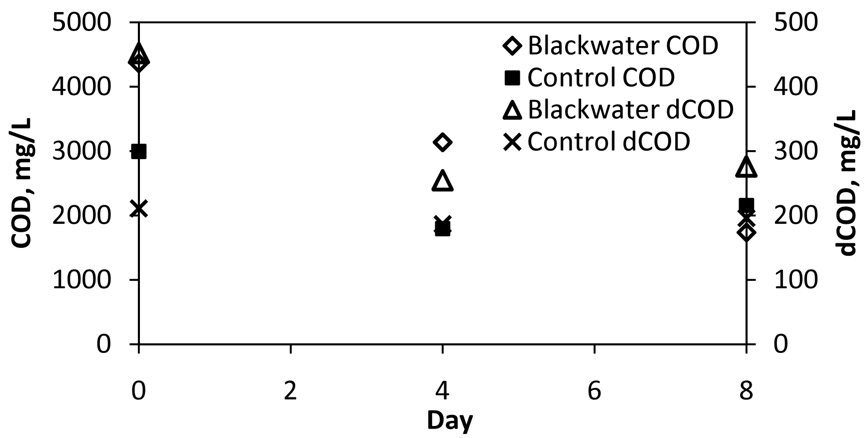
Final toxicity testing was performed examining cleaning products used for cleaning toilets and floors in the study building (Foothills Campus Atmospheric Science Chemistry Building). Inhibition was not experienced in any of the cleaning product assays and within eight days all three assays neared complete removal of dCOD (Figure 4.14). COD was also measured, however experimental error provided results unusable.

Figure 4.14: dCOD reduction in cleaning product assays during toxicity testing



Results from blackwater assays show successful biodegradation of COD and dCOD over the eight day period (Figure 4.15).

Figure 4.15: COD and dCOD reduction in blackwater during toxicity testing



Further testing ruled out all obvious causes of methanogenic inhibition or toxicity. Low biogas methane production could have been contributed to sampling error e.g. collection inefficiencies, or analytical instrument error. Also, BCMP assays were prepared using an anaerobic transfer method later ruled out as unreliable. Oxygen contamination may have cause initial toxicity to methanogens in inoculum. Finally, inoculum was taken from a waste/recycle line at a municipal facility. Depending on the age of sludge provided, sensitive methanogenic organisms may have been compromised before or during collection.

4.3.3 UASB Reactor Demonstration Study

The UASB reactor treating blackwater from the Foothills Campus was operated for a total period of 108 days. Startup of the reactor lasted until day 48 during which OLR varied from 1.18-0.29 kg COD/m^3·d at beginning and end respectively. Following startup, two phases of continuous operation are identified as Phase I (day 49-80) and Phase II (day 94-108). Phase I is broken into Phase Ia (HRT of 4.0 days) and Phase Ib (HRT of 2.6 days). A brief shutdown period and startup period breaks Phase I and Phase II. Throughout Phase II, reactor operated at an HRT of 3.7 days.

During the entire study, influent clogging reduced the actual inflow into the reactor. Influent and effluent flows were measured during Phase II of experimentation and show a mean effluent to influent flowrate ratio of 76% with a standard deviation of 6.8%. Effluent flowrate was not measured during initial experimentation or Phase I. It is assumed the same average ratio of effluent:influent occurred during Phase I. Unless otherwise noted, flowrate, HRT, and OLR values have been adjusted according to the observed ratio.

4.3.3.1 Startup

The reactor was operated at an OLR of 1.18 kg COD/m^3·d (98 mL/min) for 20 days. On day 14, washout of biomass was observed at the top of the reactor (Figure 4.16). The reactor was monitored over the next six days and washout problems had declined slightly but were still evident. On day 20, the reactor loading was lowered to an OLR of 0.29 kg COD/m^3·d (24 mL/min) to reduce washout. After noticeable decline in visible biomass washout, the reactor was sampled on day 23, 28, and 35 of operation for influent and effluent COD and dCOD. Removal efficiency of both COD and dCOD increased over the course of the sampling period from 61% to 79% and from -45% to 14% respectively (Table 4.7).

Figure 4.16: Biomass washout at the reactor surface

Table 4.7: Initial UASB sampling results

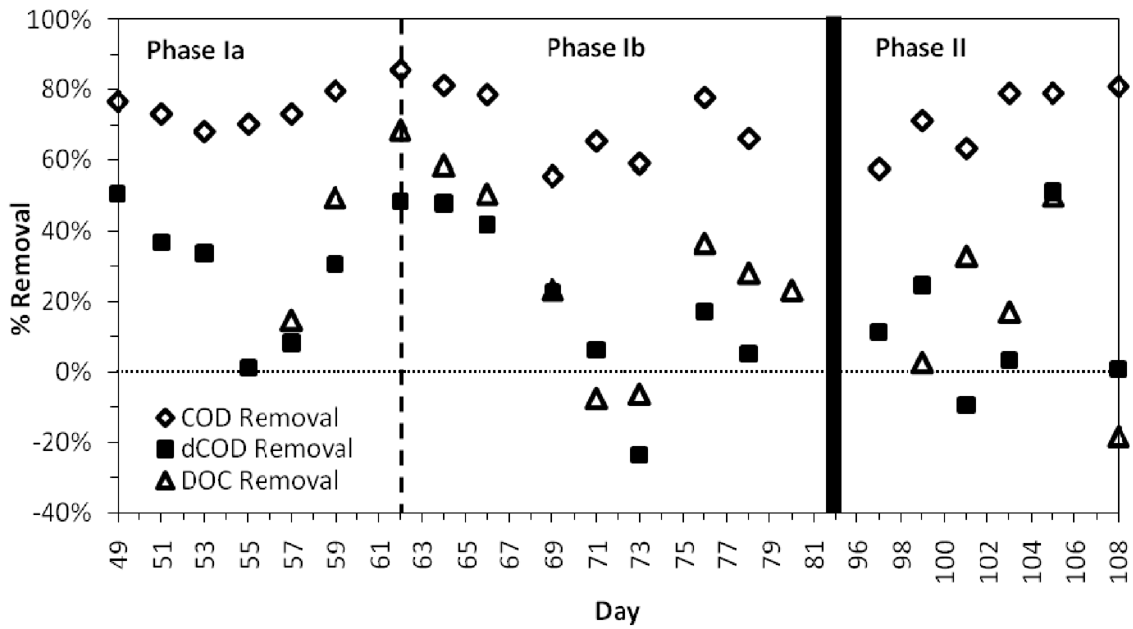
	Influent		Effluent		Removal Efficiency	
Day	*COD*	*dCOD*	*COD*	*dCOD*	*COD*	*dCOD*
-	(mg/L)	(mg/L)	(mg/L)	(mg/L)	%	%
23	975	204	378	296	61%	-45%
28	903	210	320	227	65%	-8%
35	1580	199	337	171	79%	14%

Negative dCOD removal efficiency is not uncommon and has been contributed to a combination of unutilized dCOD from influent in addition to particulate COD hydrolyzed in the reactor (Luostarinen et al. 2007). Clogging was a major problem with the designed inlet during experimentation. Toilet paper and other solids had to be removed from inside the cap on a daily basis. In hindsight, an open pipe covered above by a circular shield with maximum open space could have been used for influent distribution to prevent clogging.

4.3.3.2 Phase I & II Monitoring

A mean COD removal rate of 72% was achieved over the entire study period (Phases I & II). The mean dCOD removal over the study period was substantially lower at 21%. These values are in line with performance efficiencies in previously studied blackwater digesters. Highest COD removal occurred during Phase Ia (75% COD and 30% dCOD). A decline in performance is visible after an increase in reactor OLR occurred at the beginning of Phase Ib. Around day 75, the reactor began to recover and treatment performance improved (Figure 4.17). Unfortunately, the reactor had to be shut down for a short period after day 80 and was not able to recover completely.

Figure 4.17: Reactor COD, dCOD, and DOC removal performance



During several sampling events throughout operation, effluent dCOD was higher in concentration than influent dCOD. This increase shows that hydrolysis of particulate material was occurring. However, rate limiting is occurring in one of the following steps, acidogenesis, acetogenesis, or methanogenesis (Kujawa-Roeleveld 2005) preventing more complete utilization of soluble COD.

Great difficulty was encountered with biogas volume measurement. Reliable volume production data is only available for one sampling period. Three different apparatuses were used for gas volume measurement over the period of testing. The first apparatus was an accumulation system installed on day 55 which remained in place until the end of sampling Phase II. This device was sufficient until the reactor and device were resealed with silicone caulk.

At this time, gas collection increased substantially and produced greater than measureable quantities than the device could accommodate. A new device was installed and operation began on day 94. Gas was effectively collected between day 94 and day 97. At day 97 the line became pinched. The pinch was fixed on day 99, however the pinched line caused buildup of gas in the reactor and reactor seals breached causing low gas volume and methane concentration collection thereafter.

After failure of the second gas collection effort, the reactor was resealed and a final measurement device was installed. This device consisted of a three valve system using liquid displacement in a graduated cylinder (Figure 4.8). Methane production volume was measured over a period of 2 hours and displacement recorded. Readings from this device were greatly dependent on equilibrium in the liquid level and pressure in the reactor because of the small liquid surface area in the 200 mL graduated cylinder used. For this reason these readings have been disregarded. The methane gas volume collected between day 94-97 in the second accumulation system will be used for gas production potential purposes.

Actual and theoretical (Tchobanoglous et al. 2003) methane production volumes were calculated for this period. Using the measured effluent flowrate of 16.7 L/d for this period, a methane production rate of 0.20 L CH_4/L BW_{inflow} was calculated. Theoretical daily gas production was calculated in accordance with common design methods (Tchobanoglous et al. 2003) and study period mean COD influent and effluent was used for calculations. The same flow rate was used to calculate both actual and theoretical values. Theoretical calculations using mean study COD and blackwater flowrate yield a value of 0.21 L CH_4/L BW_{inflow} produced at this flowrate, comparable to actual results. These yields were low as compared to BCMP test results of 0.43 L CH_4/ L BW_{inflow} because COD concentrations in the blackwater were much less (901 mg COD/L compared to 1,983 mg COD/L).

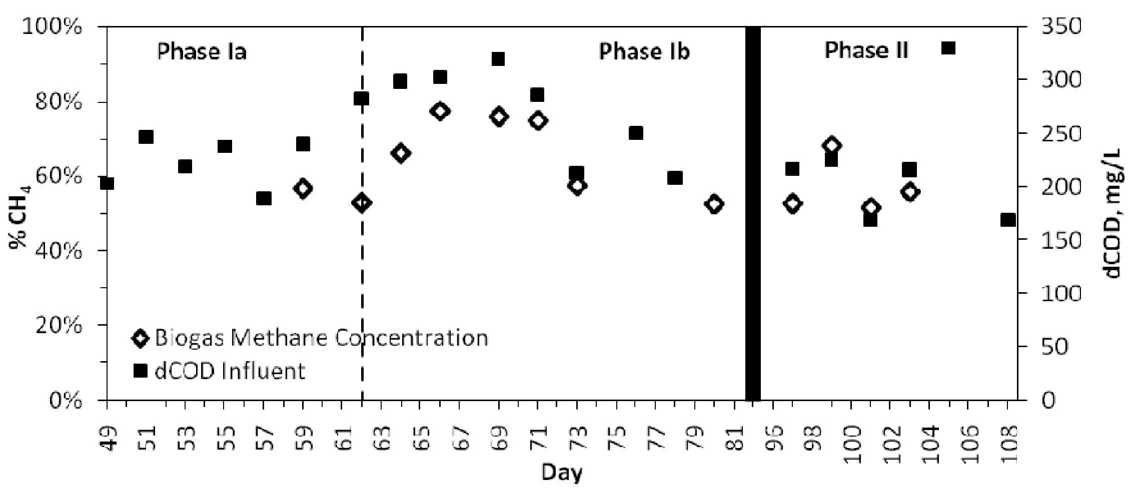
Methane volume was also calculated as a function of COD_{input} and $COD_{removed}$ in the reactor (Table 4.8). This data was compared to similar calculations as observed in another study where blackwater was treated in an anaerobic digester (Wendland 2009). Methane production in this study was somewhat less due to the lower COD influent concentration (901 mg COD/L compared to 8,060 mg/L) and OLR (0.27 kg COD/m^3·d compared to 0.45 kg COD/m^3·d).

Table 4.8: CH_4 production normalized to COD input and COD removed

	Wendland (2009)	CSU Blackwater
L CH_4/kg COD_{input}	209	137
L CH_4/kg $COD_{removed}$	342	192

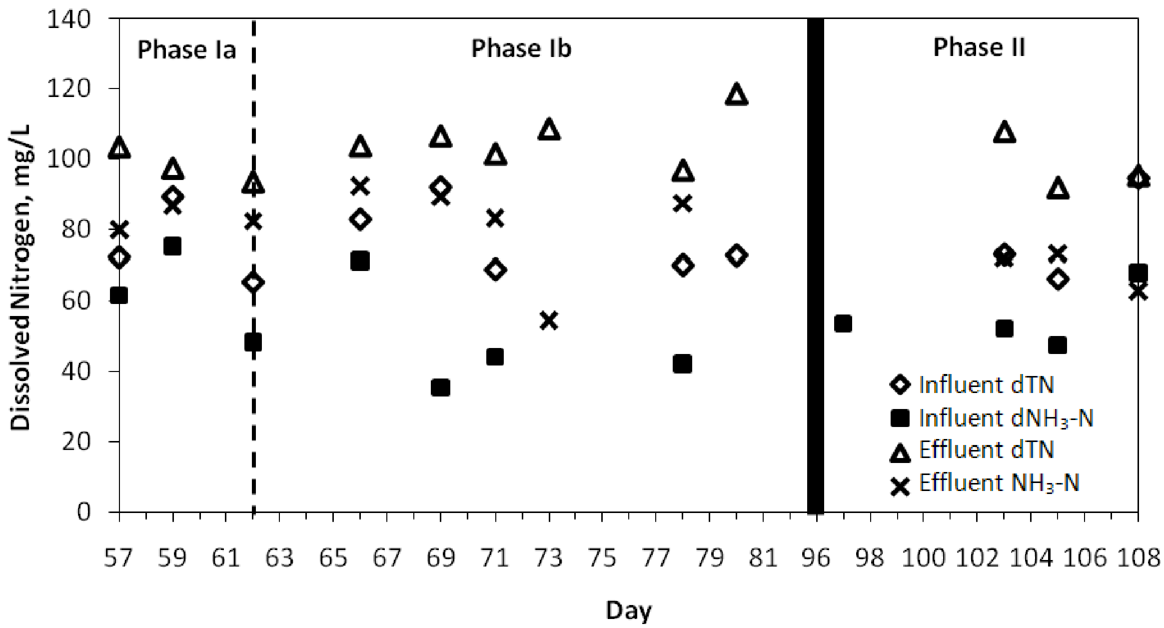
There is a visible correlation between methane concentration and dCOD influent trends (Figure 4.18). It is likely that increased soluble influent was more readily consumed by methanogens causing this phenomenon. Mean biogas methane concentration over the study period was 62%.

Figure 4.18: Reactor biogas methane concentration



Mean effluent dTN ranged from 29-38% higher than influent dTN concentration during reactor monitoring with an overall mean of 35%. Two sources have been identified for the increase in effluent dTN including hydrolysis of particulate material and release of soluble microbial product from endogenous decay. Similar increase in dTN was experienced during anaerobic biodegradation batch tests (Figure 4.19).

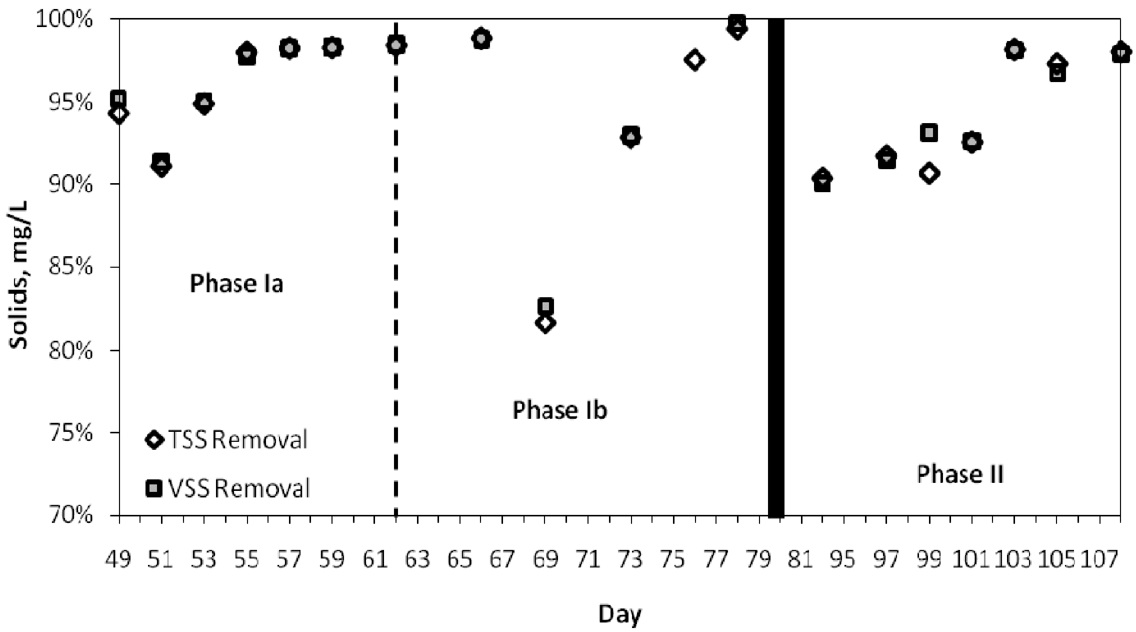
Figure 4.19: Reactor total nitrogen and ammonia concentrations



The ratio of mean dNH_3-N:dTN was higher in effluent (78%) than in influent (71%) over the entire monitoring period. Higher effluent dNH_3-N values can be contributed to hydrolysis and further conversion of urea to ammonia.

Solids removal in the reactor was maintained above 90% for the duration of monitoring with the exception of day 69 measurements where TSS and VSS removal dropped to 82% and 83% respectively (Figure 4.20). Mean effluent TSS over the study period was 18.8 mg/L with a standard deviation of 15 mg/L. Florida and Washington have set TSS standards for agricultural reuse on food crops at 5.0 mg/L and 30 mg/L respectively (USEPA 2004). The values obtained in this reactor are very close to the Washington state reuse standard for TSS. Reactor loading was increased on day 62 from 0.21 to 0.39 kg COD/m^3·d. The drop in effluent solids quality on day 69 could have been caused by washout of biofilm or attached solids in the exit weir or piping. It is also possible that a spike in influent solids concentration was experienced during this time. The ratio of VSS:TSS in reactor effluent (86.7%) was slightly less than the influent ratio (90.5%). Although the ratios are similar, VSS was reduced by a mean of 95%, showing good utilization of organic matter by microbes.

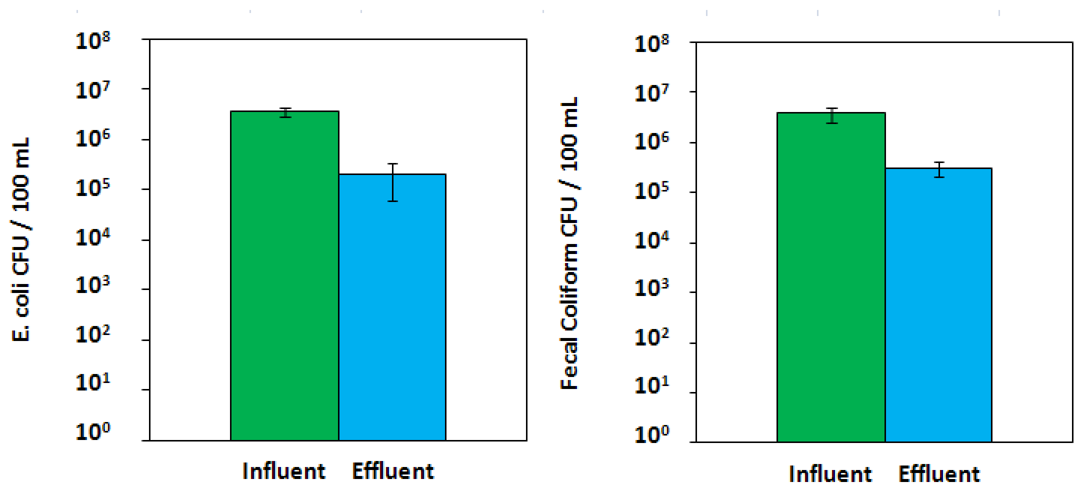
Figure 4.20: Reactor TSS and VSS removal efficiency



Five days of E. coli and fecal coliform influent and effluent values were analyzed for mean colony forming units (CFU) per 100 mL. Influent E. coli and fecal coliform data from day 97 and effluent fecal coliform data from day 105 were not included in calculations because the measured values were over the measureable range. E.coli and fecal coliforms were reduced substantially in the reactor at 1.4 log (94%) and 1.1 log (92%) respectively (Figure 4.21).

Discharge of E. coli and fecal coliforms is not regulated by the EPA for centralized waste treatment (USEPA 2001). However, the EPA has set a stringent reuse guideline for fecal coliforms of 0 CFU/100 mL in 90% of samples and 14 CFU/100 mL for any one sample (USEPA 2004). The World Health Organization (WHO) has set standards for fecal coliforms and E. coli of

Figure 4.21: Influent and effluent E. coli (left) and fecal coliform (right) measurements



less than 1000 CFU/100 mL for unrestricted irrigation of edible crops, sports fields, and public parks (Mara & Cairncross 1989). Many U.S. States including California, Nevada, Hawaii, Texas, and Washington have set food crop agricultural reuse standards for average total coliforms ranging between 2.2 CFU/100 mL and 200 CFU/100 mL (USEPA 2004). Measured effluent fecal coliform concentrations do not meet these standards. However, incorporation of a low maintenance disinfection system such as UV light would reduce effluent E. coli and fecal coliform CFU even further with little additional increase in operations efforts.

Mean influent temperature during Phase I was 17°C. Mean influent dropped to 13°C during Phase II, which can be contributed to cold flushwater caused by low outside winter temperatures. The reactor temperature controller was set at 34°C throughout the duration of the study. Measured mean effluent temperature remained constant at 28°C throughout Phase I and Phase II. During Phase I of experimentation, influent pH ranged between 8.6-9.2 with a mean of 8.9 and effluent pH ranged between 7.1-7.4 with a mean of 7.2. Phase II of experimentation had a mean influent pH of 9.1 and ranged between 9.1-9.2. Phase II effluent pH ranged between 7.5-7.8 with a mean of 7.6. High influent pH was contributed to the hydrolysis of urea to ammonia.

4.4 Preliminary Economic Analysis of Treatment Alternatives

An economic analysis has been performed comparing costs for onsite and offsite treatment of wastewater produced on the Foothills Campus. Wastewater production volumes were taken from development Scenario IV in Chapter 2, estimated flows from the Foothills Campus at complete buildout. In Scenario IV, wastewater flow volumes were calculated for source separated streams including blackwater, graywater, and labwater. In estimation of costs for aerobic onsite and offsite treatment, total wastewater production (blackwater, graywater, and labwater) was used for calculations, assuming combined wastewater will be necessary to provide sufficient dilution. In estimation of anaerobic onsite treatment costs, only blackwater volume was used, as graywater will be sent for wetland treatment which does not require operational energy inputs.

Costs related to onsite treatment were determined based solely on operational inputs. Costs were estimated for both aerobic and anaerobic onsite treatment. Costs related to offsite treatment were estimated using the City of Fort Collins commercial rate for wastewater treatment (Criswell & Roesner 2005). Estimation of treatment costs for onsite systems incorporate fundamental energy inputs and salary of one operations technician. Aerobic onsite treatment cost inputs include energy input for aeration and influent pumping. Anaerobic onsite treatment cost inputs include influent wastewater heating, recovery of heat losses, and influent pumping. Methane production from anaerobic onsite treatment was incorporated as an energy output. Energy inputs were valued using a three year average energy cost charged to CSU for Foothills Campus energy demands.

In this comparison, maintenance costs for onsite systems were limited to the cost to support the salary of a highly qualified wastewater operations technician. As discussed with CSU Department of Facilities Management, maintenance resources are not an issue of concern. This view may change over time and depending on the complexity of and liability assumed for onsite systems, it may be necessary to hire and train additional specialized operations staff. Also, when further details are decided on desired system design, a detailed economic analysis can be performed examining all operational inputs including operator training, and additional energy or material requirements.

Capital costs are not included in this comparison. The City of Fort Collins has placed a discharge limit on Foothills Campus wastewater volume. The Foothills Campus is nearing this limit. Once the limit is tripped, the City of Fort Collins will require CSU to make improvements to their wastewater infrastructure most likely to include a new large interceptor and updates to their treatment facility. An assumption was made, that the costs to construct infrastructure required by the City of Fort Collins can be normalized with the costs to construct a decentralized wastewater conveyance and treatment architecture on the Foothills Campus. Further analysis of capital costs for onsite and offsite treatment is absolutely necessary to determine the true value of capital costs in each scenario. However, the breadth of this study required that normalization of capital costs was assumed.

This is a fundamental comparison of costs, the methods used to determine costs are clearly presented. It is important to note that this is not a complete comparison on any level. However, the results of this analysis are intended to provide insight into magnitudes of costs for onsite and offsite treatment on a preliminary level. It is also important to note that if Colorado State University is to construct decentralized treatment facilities on their Foothills Campus, liability for these systems becomes their responsibility unless transferred accordingly.

4.4.1 Cost Inputs

4.4.1.1 Maintenance and Operations

To determine minimal maintenance and operations costs, the cost to support one plant operations technician was calculated. An annual salary of $61,000 was used for calculation purposes (City of Broomfield 2010). This salary is the high end of the salary range for Class A operators in the City of Broomfield, Colorado. Class A represents the highest trained class of wastewater operators as licensed by the Colorado Department of Public Health and Environment.

4.4.1.2 Energy

A three year average energy unit cost for the Foothills Campus was determined using annual average usage and cost (Table 4.9) (CSU 2010). This energy cost value was used to determine costs for all energy inputs and outputs.

Table 4.9: Determination of Foothills Campus unit energy cost

	Year	**Usage**	**Cost**	**Rate**	**3-Year Average Rate**
		KWH	$	$/KWH	$/KWH
Foothills Campus	2008	218,960,062	$13,372,357	$0.061	$0.069
Foothills Campus	2009	341,919,761	$25,168,409	$0.074	
Foothills Campus (YTD)	2010	13,750,164	$986,188	$0.072	

It is important to note that the actual value of methane will reflect efficiencies of processes used to convert the gas into energy or heat. This calculation method assumed 100% efficiency, which will need to be adjusted when appropriate application is chosen e.g. fuel for a boiler or an engine.

4.4.1.3 Influent Blackwater Pumping

Pump sizing was performed using Myers Automated Pump Selection tool (FEMYERS 2010). Flowrate used for pump sizing was based off a daily intermittent pump operation of 6 hours (Table 4.10).

Table 4.10: Parameters used for influent pump sizing

	Inflow	Flow Variation Factor	Design Flow	System Head
	m^3/day		m^3/day	m
Anaerobic Onsite	253	4	1,012	7.5
Aerobic Onsite	2,732	4	10,929	7.5

For influent pumping, anaerobic and aerobic onsite treatment required 2.2 kW and 12.4 kW motors respectively. To determine energy input, it was assumed each pump would operate for six hours per day.

4.4.1.4 Aeration

Aeration energy input was calculated using coarse bubble diffuser energy requirement of 56.4 kJ/s·1000 m^3 (Owen 1982).

4.4.1.5 Methane Production

Methane production was calculated using an experimental rate from this study of 0.2 m^3 CH_4/m^3 BW and a lower heating value of 35,800 kJ/m^3 (Tchobanoglous et al. 2003) resulting in a rate of 192 L CH_4/kg COD destroyed. A methane production rate of 342 L CH_4/kg COD destroyed from a previous study was also used as a comparison (Wendland 2009). Energy output was determined assuming the reactor would produce an equal volume of methane daily.

4.4.1.6 Heating Requirements

Two types of reactor heating requirements were determined, energy required to heat influent blackwater and energy required to account for heat losses through the reactor body. Calculations were performed assuming a mesophilic operational temperature of 35°C.

Heat losses were calculated as:

$$Q_d = C \times A(T_{reactor} - T_{outside})$$

Where:

Q_d = Energy input required to satisfy heat losses

C = Heat flow coefficient

A = Reactor surface area

$T_{reactor}$ = Reactor temperature

$T_{outside}$ = Outside temperature

Reactor material used for determination of heat flow coefficients was 30.5 cm thick insulated concrete walls, 30.5 cm thick uninsulated concrete bottom, and sealed steel cover. Heat losses were calculated for each season using Fort Collins average seasonal temperatures (WRCC 2010).

Influent heating requirements were calculated as:

$$Q_s = W \times C_s(T - T_i)$$

Where:

Q_s = Energy input required to raise influent to reactor temperature

W = Mass flow of influent blackwater

C_s = Specific heat constant of water (4,200 J/kg·°C)

T = Temperature of digester

T_i = Temperature of influent blackwater

4.4.2 Results

4.4.2.1 Municipal Treatment Costs

Commercial wastewater rates are based off potable water consumption and vary depending on the meter size serving development. The number of meters necessary to serve

Foothills Campus development at buildout was not known and will require detailed analysis and negotiation with the City of Fort Collins. For 4” and 6” meters, the annual base charge is in the range of $2,000 and $8,000 respectively. Meter base charges were not included in calculations, and depending on the number of meters could add a considerable amount to the total offsite wastewater treatment costs. Only annual variable production charges were used to calculate the cost of offsite wastewater treatment per unit volume discharged. Discharge volume was based off total wastewater estimations determined in the planning portion of this study. At a wastewater generation of 2730 m^3/day, total annual cost to route wastewater flows offsite is estimated at $695,500 (Table 4.11).

Table 4.11: Annual offsite treatment costs excluding water meter base charge

Annual Production	**Annual Production Charge**
m^3	$
997,000	$695,000

4.4.2.2 Aerobic onsite treatment costs

Support for one plant operations technician, air diffusion pumping, and influent blackwater pumping energy inputs were accounted for in determination of aerobic lagoon costs (Table 4.12).

Table 4.12: Aerobic onsite treatment cost estimation

Maintenance	**Aeration**	**Influent Pumping**	
Technician Salary	**Energy Input**	**Energy Input**	**Total Cost**
$/year	KWH/year	KWH/year	$/year
$61,000	1,350,000	27,000	$156,000

4.4.2.3 Anaerobic onsite treatment costs

Anaerobic treatment costs were determined according to support for one plant operations technician, energy inputs for influent heating, reactor heat losses and influent pumping, and energy output from methane generation (Table 4.13).

Table 4.13: Anaerobic onsite treatment cost estimation

		Main-tenance	**Influent Heating**	**Heat Losses**	**Influent Pumping**	**Methane**	
Study	**CH4 Production Rate**	**Tech-nician Salary**	**Energy Input**	**Energy Input**	**Energy Input**	**Energy Gene-rated**	**Total Cost**
	LCH_4/kgCOD destroyed	$/year	KWH /year	KWH/ year	KWH/ year	KWH /year	$/year
This Study	192	$61,000	1,616,000	36,000	5,000	254,000	$158,000
Wendland (2009)	342					452,000	$144,000

4.4.3 Conclusions

The economic comparison of alternatives shows that, using restricted cost inputs discussed for onsite treatment options, onsite treatment costs show promise to provide innovative and sustainable treatment of Foothills Campus wastewater (Table 4.14).

Table 4.14: Summary of cost estimation results

Aerobic Onsite Treatment	Anaerobic Onsite Treatment	Offsite Treatment
$151,000	$156,000	$695,000

Future economic work must involve more detailed comparison of operations, maintenance and capital costs for onsite and offsite management options. Operations and maintenance costs should focus on a particular onsite treatment technology, e.g. anaerobic treatment, and determine costs of detailed inputs, outputs, and additional processes necessary, e.g. disinfection, to provide sufficient treatment for chosen end use, e.g. irrigation. These costs will provide closer comparison of onsite treatment costs to the City of Fort Collins Utilities commercial wastewater rates, which likely include, operations, maintenance, and capital costs.

Because CSU will encounter additional capital costs with either onsite or offsite wastewater management, a comparison of capital costs should be developed to compare costs for an onsite treatment architecture and costs diverted to the City of Fort Collins if offsite treatment is chosen, e.g. interceptor construction and plant expansion costs. It is important to note that if Colorado State University is to construct decentralized treatment facilities on their Foothills Campus, liability for these systems becomes their responsibility unless transferred accordingly.

4.5 Conclusions

Overall, the UASB reactor provided effective treatment of blackwater from the Foothills Campus. Stable operation was experienced throughout the study and the reactor recovered quickly from changes in OLR. High buffering capacity of blackwater prevented pH from leaving the acceptable range during the study period. Removal of COD was comparable to previously performed studies and solids reduction was exceptionally high (Table 4.15). Indicator organisms E. coli and fecal coliforms were reduced by >1 log reduction (1.4 log and 1.2 log respectively) without further disinfection. Disinfection would be necessary to meet regulatory requirements and recommendations put forth by the EPA and other U.S. State agencies.

Methane production was lower than experienced in BCMP testing due to lower substrate COD concentrations. However, because the UASB was limited by pumping diameter, full scale operation would receive COD concentrations comparable to BCMP tests resulting in greater volumes of methane. It is also likely that ineffectiveness of methane volume collection in this study led to error in methane production values. A more accurate and reliable method for monitoring methane flowrate should be incorporated in future studies. The carbon cycle was not closed in this study. Therefore it was not possible to calculate experimental error. Frequent clogging of the influent distribution required daily observation and unclogging. A better design should be incorporated to prevent frequent maintenance.

Table 4.15: Mean experimental results over entire study period (Phase I & II)

Parameter	Unit	UASB Influent		UASB Effluent		Removal Rate	
pH	---	8.9	(0.3)	7.4	(0.3)	-	-
Temperature	°C	16	(2.1)	28	(2.5)	-	-
COD	mg/L	901	(215)	258	(102)	72%	(8.6%)
dCOD	mg/L	238	(49)	186	(42)	21%	(22%)
DOC	mg/L	53	(15)	37	(13)	27%	(25%)
TSS	mg/L	435	(147)	19	(15)	95%	(4.5%)
VSS	mg/L	379	(119)	17	(14)	95%	(4.3%)
dTN	mg/L	77	(11)	102	(7.7)	-34%	(19%)
dNH_3-N	mg/L	54	(13)	79	(12)	-58%	(48%)
E. coli	CFU/100 mL	$3.6x10^6$	$8.0x10^5$	$2.0x10^5$	$1.4x10^5$	1.4 log	(4.1%)
Fecal coliforms	CFU/100 mL	$3.8x10^6$	$1.3x10^6$	$3.1x10^5$	$1.0x10^5$	1.1 log	(4.1%)
CH4	L CH_4/kg COD_{input}	137					
CH4	%	62%					

*Standard deviations are shown in parenthesis

Biogas production and effluent nutrient concentrations show potential for reuse applications. Further study is recommended to determine the potential and feasibility of direct irrigation with disinfected effluent. Characterization of nutrient, metal, and VFA concentrations should be performed to determine suitability for agricultural application. Use of alternative substrates to supplement blackwater should be explored including organic kitchen refuse and animal manure, both readily available from CSU. Reactor sizing and performance could benefit from the use of low flush or vacuum toilets. These technologies should be explored. Preliminary economic comparison of alternative onsite and offsite treatment alternatives showed promise for onsite treatment to serve as an innovative and sustainable treatment option for the Foothill Campus.

5.0 SUMMARY CONCLUSIONS

Advanced decentralized wastewater management (ADWM) concepts for the Colorado State University (CSU) Foothills Campus have been investigated through planning and demonstration. A planning study presented four management scenarios for ADWM of wastewater from proposed and existing development on the Foothills Campus. A multi-criteria decision analysis (MCDA) was performed to select the most appropriate technology for decentralized treatment of blackwater from the Foothills campus. Based on CSU specific criteria, an upflow anaerobic sludge blanket (UASB) reactor was selected. A pilot project was conducted utilizing a UASB to treat raw blackwater from the Foothills Campus. The UASB reactor was operated over a period of 108 days. Substantial reductions were achieved in effluent COD (72%), solids (95%), and indicator organism concentrations (1.4 log E. coli, 1.1 log fecal coliforms).

5.1 Foothills Campus Sustainable Wastewater Management Planning Study

Developing a plan to provide ADWM on the Foothills Campus is a crucial undertaking to avoid future expansion fees imposed on CSU by the City of Fort Collins Utilities. Incorporation of ADWM concepts into the plan puts CSU at the forefront of innovation in sustainable wastewater management. Management of wastewater from a research campus setting is inherently complex due to the heterogeneous mix of wastewater sources, for example water form laboratories which has potential to contain biological or chemical contaminants at levels which could provide upset to biological treatment processes and present concerns with human contact in water reuse systems.

Four wastewater management scenarios were developed which incorporate utilization of existing conveyance and collection infrastructure. Options for management of wastewater from existing buildings was considered separately than for new development. Combined plumbing of existing development is maintained in all scenarios to avoid prohibitive modification costs. Three options for management of existing development wastewater were presented. Recommendations for management of wastewater from existing development wastewater were:

- Settle solids, combine solids with proposed development blackwater and untreated liquid with proposed development graywater
- Utilize existing infrastructure to route flow to an 'onsite centralized' treatment facility located on the Foothills Campus
- Continue to route flow through existing infrastructure offsite for treatment

Wastewater from proposed development was separated into four characteristic quality streams in each scenario identified as graywater, blackwater, lab process water, and lab sink water. The treatment technologies considered for graywater and blackwater were constructed wetlands and anaerobic digesters, respectively, to maximize extraction of valuable resources (reusable water, nutrients, and biogas). Lab process water, originating from laboratory processes form which water does not have potential to come in contact with harmful contaminants, e.g. autoclave cooling water, can be combined with graywater for wetland treatment. Lab sink water should be separated due to the potential for this stream to contain shock loadings of chemical or biological contaminants which could upset biological treatment processes and cause concern with human contact in reuse systems. Lab sink water can either be sent offsite through existing infrastructure, or combined with existing wastewater for treatment onsite. If treated onsite, appropriate pretreatment or innovative monitoring systems are recommended for lab sink water to prevent shock contaminant loadings. In addition to treating graywater, wetland treatment could be utilized for effluent liquid from a blackwater treatment process to provide higher quality water for end use, e.g. irrigation.

CSU will expand the total square footage of Foothills Campus buildings from near 1.0 MSF to roughly 3.5 MSF at complete buildout over approximately 25 years. The value of methane produced from proposed development blackwater at buildout was estimated above $4 million if all biogas produced was used as a direct supplement for natural gas. Flows from existing development on the Foothills Campus are reaching the discharge limit set by the City of Fort Collins.

Incorporating onsite wastewater management architecture to capture proposed development, similar to Scenario II presented in this study, provides a solid starting point to remove the Foothills Campus from the municipal wastewater system. Eventual transition to capture and treat existing development, similar to Scenario I, will allow CSU to completely remove the Foothills Campus from the municipal wastewater system and be completely self reliant. Scenarios I & II in presented, or modified form to capture lab sink water, provide the most advantageous opportunity for CSU to incorporate innovative and sustainable concepts, while providing the option for redundancy if cluster treatment facilities are connected.

As development continues, alternative approaches to management of wastewater from the Foothills Campus will be necessary to prevent additional fees for expansion of Fort Collins municipal infrastructure. By incorporating discussed ADWM concepts for treatment of wastewater on the Foothills Campus, CSU becomes a model for other institutions to follow, creates great potential for future research opportunities examining the future of wastewater management, and contributes greatly to increasing environmental sustainability of campus operations.

5.2 Selection of Appropriate Technology for Decentralized Treatment of Blackwater at Foothills Campus

A comparison of six reactor technologies, five anaerobic and one aerobic, found the most appropriate technology for treatment of raw blackwater from the Foothills Campus was a UASB reactor. Technologies were first compared by ranking developed criteria and sub-criteria and second subject to a weighted comparison based on the importance of each criterion to CSU for development of wastewater treatment processes on the Foothills Campus. Categories, with importance to CSU out of ten shown in parenthesis, included energy input (9), energy output (9), capital investment (8), treatment efficiency (7), operational complexity (6), and maintenance required (3). The UASB ranked highest overall due to high scores for positive energy balance, as well as comparatively low initial capital investment and maintenance requirements. The comparison performed was subjective to qualitative rankings, however, provided a treatment technology which was effective in treating blackwater from the Foothills Campus.

5.3 Anaerobic Digestion of Blackwater Study

Anaerobic digestion by means of a UASB reactor design is an effective technology for treatment of blackwater from the Foothills Campus. Study results showed that on average 70% reduction in total COD, and 95% reduction in TSS and VSS could be achieved by a UASB reactor. Operation of the reactor at a HRT between 2.6-4.0 days and OLR between 0.21-0.39 $kgCOD/m^3 \cdot d$ provided stable, uninhibited operation. Greater than one log reduction was experienced for indicator organisms E. coli (1.4 log destruction) and fecal coliforms (1.1 log destruction) without further disinfection. Relatively high biogas methane concentration (62%) and volume (137 L CH_4/kg COD_{inout}) measurement show promising results for use of biogas as a reliable source of renewable energy. Effluent dissolved nitrogen increased 34% to 102 mg/L, due to hydrolysis of particulate matter. This nitrogen is a valuable nutrient source for irrigation application and of disinfected reactor effluent as a source of irrigation applied fertilizer for non-consumptive crops grown on the Foothills Campus or combined with graywater for wetland treatment are both viable options. Preliminary economic comparison of alternative onsite and offsite treatment alternatives showed promise for onsite treatment to serve as an innovative and sustainable treatment option for the Foothill Campus.

5.4 Conclusions

Anaerobic digestion of blackwater is a feasible option for decentralized treatment of wastewater generated at the Foothills Campus. Anaerobic digestion provides substantial treatment of wastewater parameters, while producing renewable energy in the form of methane biogas and an effluent rich in reusable nutrients. Serious consideration should be placed on reuse of nutrient enhanced effluents from anaerobic treatment processes as irrigation applied fertilizer as well as using solids produced as a source of soil amendment on the Foothills Campus. Use of locally available nutrients for fertilizer saves energy by eliminating depletion of natural gas used to make nitrogen fertilizers, saves costs of transporting fertilizers, and reduces the energy required for aerated wastewater nutrient removal processes (Etnier 2007).

Further treatment e.g. filtration and pathogen destruction should be examined for blackwater effluent applied for reuse to protect public health. Examination of phosphorus, magnesium, potassium, sulfur, metals, and volatile fatty acid concentrations in treatment process effluent and solids is necessary to ensure plant health is maintained. Also, the fate of emerging contaminants (including pharmaceuticals and personal care products) in soils irrigated with treatment process effluent should be examined. Although, destruction of emerging contaminants in soils is likely more effective than in waters due to the large surface area irrigated and the complex network of microbes found in soils (which is denser than microbial populations in waterways) capable of destroying organic compounds (Etnier 2007).

Addition of locally available organic waste sources into the blackwater stream such as animal manure and organic kitchen refuse into the digester should be researched. Additional organic wastes would increase methane production, consolidate disposal of organic wastes on the Foothills Campus into one stream, and increase nutrient concentrations of reactor outputs for use as fertilizer.

Further study on performance of anaerobic digestion of blackwater from reduced flush or vacuum flush toilets should be considered as a viable option to increase solids concentration, beneficial for anaerobic digestion, and reduce water consumption. Waterless urinals should be incorporated into future development to reduce water consumption and prevent unnecessary dilution. In addition, alternative collection systems such as vacuum collection should be seriously considered for application on the Foothills Campus. Vacuum collection reduces the amount of flushwater necessary to transport waste, prevents unnecessary dilution increasing the efficiency of anaerobic digestion and reducing the size of infrastructure and treatment processes. Negative pressure piping also eliminates leaking infrastructure, enhancing environmental protection.

Development of wastewater management infrastructure for the Foothills Campus must embrace concepts which demonstrate the CSU desired aim of environmental sensitivity and sustainability (RNL Design 2003). Incorporation of innovative management concepts creates a model for other institutions to follow and provides opportunity to contribute future research on such concepts associated with ADWM. Successful design and operation of a wastewater management system on the Foothills Campus can offer financial, social, and environmental benefits to CSU and the surrounding community.

6.0 REFERENCES

Al-Jayyousi O.R. (2003). Greywater reuse: towards sustainable water management. *Desalination*, 156, 181-192.

Anderson D. and Otis R. (2000). Integrated Wastewater Management in Growing Urban Environments. In: *Managing Soils in an Urban Environment*, R. B. Brown (ed.), American Society of Agronomy, Madison, WI USA.

Anon (2009). Municipal Solid Waste Biogas Plants in China. http://www.anaerobic-digestion.com/html/msw_biogas_plants_in_china.php (accessed January 2010)

Anon (2010). National Biogas and Manure Management Programme. India Development Gateway. http://www.indg.in/rural-energy/schemes/national-biogas-and-manure-management-programme-nbmmp (accessed January 2010)

BORDA (2009). Decentralized Waste Water Treatment - DEWATS. http://www.borda-net.org/modules/cjaycontent/index.php?id=29#background (accessed January 2010)

Burian S.J., Nix S.J., Pitt R.E. and Durrans S.R. (2000). Urban wastewater management in the United States: past, present, and future. *Journal of Urban Technology*, 7(3), 33-62.

CSU (2010). Colorado State University EnergyCAP database. http://www.fm.colostate.edu /energycap/home.asp (accessed January 2010)

City of Broomfield (2010). Plant operator job announcement. http://www.broomfield.org/jobs/ jobannouncement.asp?requisition=2010-0049 (accessed March 2010)

Clesceri L., Greenberg A. and Eaton A. (1998). Standard Methods for The Examination of Water and Wastewater 20th edn, American Public Health Association/American Water Works Association/Water Environment Federation, Washington DC, USA.

Criswell M. and Roesner L. (2005). *Water Evaluation of Colorado State University's Foothills Research Campus*. MS thesis, Urban Water Center, Colorado State University, Fort Collins CO, USA.

EIA (2009). Energy Information Administration Colorado Natural Gas Pricing Index 2009. http://www.eia.doe.gov (accessed January 2009)

EPA (2009). Septic (Onsite) Systems. http://cfpub.epa.gov/owm/septic/index.cfm (accessed August 2009)

Elmitwalli T.A., van Leeuwen M., Kujawa-Roeleveld K., Sanders W. and Zeeman G. (2006). Anaerobic biodegradability and digestion in accumulation systems for concentrated black water and kitchen organic-wastes. *Water Science & Technology*, **53**(8), 167-175.

Eriksson E., Auffarth K., Henze M. and Ledin A. (2002). Characteristics of grey wastewater. *Urban Water*, **4**(1), 85-104.

Etnier C., Pinkham R., Crites R., Johnstone D.S., Clark M., Macrellis A. (2007). *Overcoming Barriers to Evaluation and Use of Decentralized Wastewater Technologies and Management*, Report, WERF, Alexandria VA, USA.

Etnier C. (2007). Research needs for decentralized wastewater treatment in an energy constrained future. *Water Environment Research*, **79**(2), 123-124.

FEMYERS (2010). Myers Automated Pump Selection Tool. http://www.femyers.com/software /Myers Software.html (accessed January 2010)

Fannin K. and Biljetina R. (1987). Reactor Designs. In: *Anaerobic Digestion of Biomass*, Chynoweth D. and Isaacson R. (ed.), Elsevier Applied Science Publishers LTD, Essex, England, pp. 141-169.

GTZ (2009). Introduction to ecosan (ecological sanitation). http://www.gtz.de/en /themen/8524.htm (accessed November 2009)

Gonchar J. (2007). The Helena: A Westside Story. GreenSource. http://greensource. construction.com/projects/0704_helena.asp (accessed January 2010)

Grady L., Daigger G. and Lim H. (1999). *Biological Wastewater Treatment*, 2nd edn, Marcel Dekker Inc., New York NY, USA.

Hobson P. and Wheatley A. (1993). *Anaerobic Digestion Modern Theory and Practice*, Elsevier Science Publishers LTD, Essex, England.

Hochedlinger M., Steinmüller H., Oldenburg M., Schroft J., Schweighofer P. and Plattner G. (2008). Experiences from the EcoSan full scale pilot project solarCity Linz. In: *Proceedings of the 11th International Conference on Urban Drainage*, Edinburgh, Scotland, pp. 1-10.

Kroeker E.J., Schulte D.D., Sparling A.B. and Lapp H.M. (1979). Anaerobic treatment process stability. *Journal of the Water Pollution Control Federation*, **51**(4), 718-727.

Kujawa-Roeleveld K. (2005). *Anaerobic treatment of concentrated wastewater in DESAR concepts*, Report, STOWA, Utrecht, Netherlands.

Kujawa-Roeleveld K., Fernandes T., Wiryawan Y., Tawfik A., Visser M. and Zeeman G. (2005). Performance of UASB septic tank for treatment of concentrated black water within DESAR concept. *Water Science & Technology*, **52**(1-2), 307-313.

Kujawa-Roeleveld K., Elmitwalli T. and Zeeman G. (2006). Enhanced primary treatment of concentrated black water and kitchen residues within DESAR concept using two types of anaerobic digesters. *Water Science & Technology*, **53**(9), 159-168.

La Motta E.J., Padrón H., Silva E., Luque J., Bustillos A. and Corzo P. (2008). Pilot plant comparison between the AFBR and the UASB reactor for municipal wastewater pretreatment. *Journal of Environmental Engineering*, **134**(4), 265-272.

Larsen T.A. and Gujer W. (1996). Separate management of anthropogenic nutrient solutions (human urine). *Water Science & Technology*, **34**(3-4), 87-94.

Larsen T.A., Alder A.C., Eggen R.I.L., Maurer M. and Lienert J. (2009). Source separation: will we see a paradigm shift in wastewater handling? *Environmental Science & Technology*, **43**(16), 6121-6125.

Lettinga G. and Hulshoff Pol L.W. (1991). UASB-process design for various types of wastewaters. *Water Science & Technology*, **24**(8), 87-107.

Lettinga G., Lens P. and Zeeman G. (2001). Environmental Protection Tehnologies for Sustainable Development. In: *Decentralised Sanitation and Reuse: Concepts, Systems and Implementation*, P. Lens, G. Zeeman and G. Lettinga (ed.), IWA Publishing, Padstow Cornwall, England, pp. 3-10.

Linz M. (2009). solarCity Pichling Project Information. http://www.linz.at/english/life/3199.asp (accessed December 2010)

Lopez Zavala M., Funamizu N. and Takakuwa T. (2002). Onsite wastewater differential treatment system: modeling approach. *Water Science & Technology*, **46**(6-7), 317-324.

Luostarinen S. (2005). *Anaerobic On-Site Wastewater Treatment at Low Temperatures*. PhD thesis, University of Jyväskylä, Jyväskylä, Finland.

Luostarinen S. and Rintala J. (2007). Anaerobic on-site treatment of kitchen waste in combination with black water in UASB-septic tanks at low temperatures. *Bioresource Technology*, **98**(9), 1734-1740.

Luostarinen S., Sanders W., Kujawa-Roeleveld K. and Zeeman G. (2007). Effect of temperature on anaerobic treatment of black water in UASB-septic tank systems. *Bioresource Technology*, **98**(5), 980–986.

Mara D. and Cairncross S. (1989). *Guidelines for the safe use of wastewater and excreta in agriculture and aquaculture*. Report, United Nations Environment Programme, WHO, Geneva, Switzerland.

March J.G., Gual M. and Orozco, F. (2004). Experiences on greywater re-use for toilet flushing in a hotel (Mallorca Island, Spain). *Desalination*, **164**(3), 241-247.

O'Connor G.A., Elliott H.A. and Bastian R.K. (2008). Degraded water reuse: an overview. *Journal of Environmental Quality*, **37**(5), 157-168.

OtterWasser GmbH (2003). Ecological settlement Lübeck - Flintenbreite. http://www.otterwasser.de/homee.htm (accessed January 2010)

Otterpohl R., Braun U. and Oldenburg M. (2003). Innovative technologies for decentralised water, wastewater and biowaste management in urban and peri-urban areas. *Water Science & Technology*, **48**(11-12), 23-32.

Owen W.F., Stuckey D.C., Healy Jr. J.B., Young L.Y. and McCarty P.L. (1979). Bioassay for monitoring biochemical methane potential and anaerobic toxicity. *Water Research*, **13**(5), 485-492.

Owen W. (1982). *Energy in Wastewater Treatment.* Prentice-Hall, Englewood Cliffs NJ, USA.

Pinkham R.D., Hurley E., Watkins K., Lovins A.B., Magliaro J., Etnier C. and Nelson V. (2004). *Valuing Decentralized Wastewater Technologies: A Catalogue of Benefits, Costs, and Economic Analysis Techniques*, Report, RMI, Snowmass CO, USA.

RNL Design (2003). *Colorado State University Foothills Research Campus Master Plan*, Report, RNL Design, Denver CO, USA.

Rittmann B. and McCarty P. (2001). *Environmental Biotechnology: Principles and Applications*. McGraw-Hill, New York NY, USA.

Roesner L., Qian Y., Criswell M., Stromberger M. and Klein S. (2006). *Long-term Effects of Landscape Irrigation Using Household Graywater - Literature Review and Synthesis*, Report, WERF, Alexandria VA, USA.

SUSANA (2009). Urine as fertilizer. http://www.susana.org/lang-en/case-studies/reuse/urine-fertilizer (accessed January 2010)

Souza M.E. (1986). Criteria for the utilization, design and operation of UASB reactors. *Water Science & Technology*, **18**(12), 55-69.

Tchobanoglous G., Burton F. and Stensel H. (2003). *Wastewater Engineering: Treatment and Reuse*. McGraw-Hill Science & Engineering, New York NY, USA.

UNEP (2000). International Source Book on Environmentally Sound Technologies for Wastewater and Stormwater Management. http://www.unep.or.jp/Ietc/Publications/TechPublications/TechPub-15/main_index.asp (accessed November 2009)

USEPA (2001). Centralized Waste Treatment Effluent Limitations Guidelines and Pretreatment Standards (40 CFR 437).

USEPA (2004). *Guidelines for Water Reuse*, Report, USEPA 625/R-04/108, Washington DC, USA.

USEPA (2007). *Innovation and Research for Water Infrastructure for the 21st Century - Research Plan*, Report, Office of Research and Development USEPA, Washington DC, USA.

van Voorthuizen E., Zwijnenburg A., van der Meer W. and Temmink H. (2008). Biological black water treatment combined with membrane separation. *Water Research*, **42**(16), 4334–4340.

Varis O. and Somlyódy L. (1997). Global urbanization and urban water: can sustainability be afforded?. *Water Science & Technology*, **35**(9), 21-32.

Vieira S.M. and Garcia A.D. (1992). Sewage treatment by UASB reactor - operation results and recommendations for design and utilization*.* *Water Science & Technology*, **25**(7), 143-157.

WRCC (2010). Fort Collins Colorado Climate Summary. http://www.wrcc.dri.edu/cgi-bin/cliMAIN.pl?co3005 (accessed January 2010)

Wageningen University (2009). Decentralised Sanitation and Reuse (DESAR). http://www.ete.wur.nl/UK/Projects/DESAR/ (accessed January 2010)

Wendland C., Deeoener S., Behrendt J., Toshev P. and Otterpohl R. (2007). Anaerobic digestion of blackwater from vacuum toilets and kitchen refuse in a continuous stirred tank reactor (CSTR). *Water Science & Technology*, **55**(7), 187-194.

Wendland C. (2009). *Anaerobic Digestion of Blackwater and Kitchen Refuse*. PhD thesis, Hamburg University of Technology, Hamburg, Germany.

Werner C., Panesar A., Rüd S.B. and Olt C.U. (2009). Ecological sanitation: principles, technologies and project examples for sustainable wastewater and excreta management*. Desalination,* **248**(1-3), 392-401.

Winblad U., Simpson-Hébert M., Calvert P., Morgan P., Rosemarin A., Sawyer R. and Xiao J. (2004). *Ecological sanitation,* Report, Stockholm Environment Institute, Stockholm, Sweden.

Worrell Water Inc. (2009). Living Machine®. http://worrellwater.com/ (accessed January 2010)

Zeeman G., Kujawa K., Mels A., Meulman B. and Kwant F. (2007). Full scale demonstration of vacuum collection, transport and treatment of blackwater. *Advanced Sanitation Conference*, Aachen, Germany.

7.0 APPENDIX A: FOOTHILLS CAMPUS EXISTING INFRASTRUCTURE CAPACITY

Although not entirely explained in the “Water and Wastewater Service Allocation Agreement” between CSU and City, it was interpreted for the purposes of this report that peak daily water use was calculated by dividing the peak month annually by the number of days in that month. SM1 has a peak daily discharge limit of 233,333 GPD and SM2 has a limit of 166,667 GPD. To determine the peak daily flow from average daily flow (ADF) for proposed development, it was necessary to find a wastewater peak factor for the Foothills Campus. This was performed using Foothills Campus monthly master meter data. Monthly master meter data used for calculations was the entire Foothills Campus master meter minus flows from Hughes Stadium. This data was taken to represent the combined flow from SM1 and SM2 and was used solely for calculation of a peak factor.

Peak daily flow from 2007 was calculated by dividing the 2007 peak month (July) by the number of days in that month (31). This results in a peak daily flow value of 254,290 GPD for the entire Foothills Campus. The 2007 ADF for existing Foothills Campus buildings was 179,168 GPD. Dividing peak daily flow by ADF, a peak factor of 1.42 was determined. This peak factor was used to estimate peak daily flows for proposed development from calculated ADF.

From information provided by the CSU Department of Facilities Management, Phase I is a 10 year development period beginning from present. Phase II represents development which will be completed 20+ years from present. For the purposes of this report, it was assumed that Phase I will last ten years from 2008-2017 and Phase II will last 15 years from 2017-2032.

Peak flow values were calculated by multiplying calculated average daily flow (ADF) for each phase of development by the peak factor of 1.42. ADF flows and calculated peak daily flows are provide in Tables 7.1 and 7.2 respectively.

Table 7.1: Average daily flow values for SM1 & SM2

	Existing ADF [GPD]	Phase I ADF [GPD]	Phase II ADF [GPD]
SM1	96,966	48,639	119,581
SM2	99,152	101,475	310,256

Table 7.2: Calculated peak daily flow values for SM1 & SM2

	Existing Peak Daily Flow [GPD]	Phase I Peak Daily Flow [GPD]	Phase II Peak Daily Flow [GPD]
SM1	137,692	69,067	169,805
SM2	140,796	144,095	440,563

Information on when individual buildings would be built within either phase was unavailable. For this reason it was necessary to spread development over each phase to predict when SM1 and SM2 would reach capacity. The baseline flow for each sewer main began with existing flow values. From this point, flows from Phase I were evenly spread over the Phase I development period. This was performed by dividing the total peak flow contribution from Phase I by 10 and adding this value to the total flow each year. The same was done for Phase II flows over the 15 year period. This method of calculation assumes that development will occur evenly over each phase. Although this is not entirely likely, it was the best possible prediction that can be made using available data.

8.0 APPENDIX B: MCDA SUPPLEMENT

Table 8.1: Summary of criteria ranking values used in MCDA analysis

	Suspended Bed	Sludge Blanket	Anaerobic Filter	Complete Mix w/ Recycle	Anaerobic MBR	Aerobic
Energy Input						
Influent Pumping	5	5	5	5	1	5
Mixing	5	5	5	1	2	5
Recycle Pumping	1	5	5	1	1	5
Aeration	5	5	5	5	5	1
TOTAL	16	20	20	12	9	16
RANK	**3**	**1**	**1**	**5**	**6**	**3**
Energy Output						
Methane Production (Rank)	1	1	1	1	1	6
Operational Complexity						
Probability of Upset	2	2	2	2	2	5
Probability of Washout	3	3	5	3	5	5
Complexity of Adjustments	2	4	5	2	1	5
Complexity of Startup	3	4	2	5	1	5
TOTAL	10	13	14	12	9	20
RANK	**5**	**3**	**2**	**4**	**6**	**1**
Maintenance Required						
Frequency of Clogging	5	5	1	5	2	5
Frequency of Sludge Removal	5	5	5	3	2	1
Frequency of Moving Part Replacment	5	5	5	3	3	5
TOTAL	15	15	11	11	7	11
RANK	**1**	**1**	**3**	**3**	**6**	**3**
Capital Investment						
Volume Necessary	5	5	5	5	3	3
Footprint Required	5	5	5	3	4	2
Material Cost	3	5	2	3	1	5
TOTAL	13	15	12	11	8	10
RANK	**2**	**1**	**3**	**4**	**6**	**5**
Treatment Efficiency						
Ability to Treat High Solids Waste	5	5	1	5	5	3
Ability to Treat Particulate COD	4	4	3	4	5	5
TOTAL	9	9	4	9	10	8
RANK	**2**	**2**	**6**	**2**	**1**	**5**

Table 8.2: Literature treatment performance data

	Substrate	CODin	OLR	Solids Loading	HRT	Temp	COD LOAD	COD Reduction	TSS Reduction	CH4 Production	CH4 Production	Flowrate	Equivalent Reactor Volume	Actual Reactor Volume
	-	Mg/L	Kg COD /m^3·d	Kg TSS /m^3·d	days	°C	Kg/m^3	%	%	% in Biogas	L_{CH4}/kg $COD_{removed}$	m^3/d	m^3	m^3
Complete Mix	**Blackwater**	**8700**	0.5	0.26	20	37	10.00	61%	51%	76%	393	-	-	-
Sludge Blanket	**Municipal**	**341**	5.5	1.2	0.133	20	0.73	34%	37%	60%	235	3.0	0.19	-
	Synthetic	**4000**	17.0	-	0.24	37	4.08	96%	-	55%	337	0.0358	0.0084	-
	Municipal	**707**	2.3	-	7.5	-	16.98	73%	-	-	-	28.8000	9.0000	9.0000
	Municipal	**712**	1.9	-	9	-	17.09	79%	-	-	-	24.0000	9.0000	9.0000
	Blackwater	**10900**	0.382	-	29	25	10.90	78%	-	-	-	0.0070	0.2000	0.2000
	Blackwater	**12311**	0.4	-	29	25	12.31	78%	-	-	-	0.0068	0.2000	0.2000
	Blackwater	**1139**	2.28	-	0.5	37	1.14	91%	-	-	104	0.01	0.0050	-
Suspended Bed	**Municipal**	**301**	3.2	0.89	3.2	24	10.24	22%	32%	57%	308	3.0	0.28	-
	Synthetic	**4000**	17	-	0.24	37	4.08	86%	-	55%	376	0.0358	0.0084	-
Fixed/Packed Bed/Filter	**Municipal**	**400**	0.7	0.68	0.5	Ambient WW	0.35	95%	95%	65%	350	0.240	0.1371	0.1200
			2.6	-	0.125		0.33	85%	-	-	-	0.960	0.1477	0.1200
			0.98	-	0.33		0.32	90%	-	-	-	0.364	0.1486	0.1200
			0.47	-	0.75		0.35	95%	-	-	-	0.160	0.1362	0.1200
			0.25	-	1.5		0.38	97%	-	-	-	0.080	0.1280	0.1200
			0.1	-	4		0.40	97%	-	-	-	0.030	0.1200	0.1200
			0.05	-	8		0.40	97%	-	-	-	0.015	0.1200	0.1200
	Municipal	**288**	0.3	0.112	1.05	20-35	0.29	73%	73%	67%	165	0.01584	0.017	-
	Synthetic	**1500**	0.85	-	0.75	25	0.64	92%	-	73%	397	0.016	0.03	-
Anaerobic MBR	**Blackwater**	**1139**	2.28	-	0.5	37	1.14	86%	-	-	131	0.0080	0.0040	-
Aerobic MBR	**Blackwater**	**1139**	2.73	-	0.42	22.5	1.15	91%	-	-	-	0.0096	0.0040	-

9.0 APPENDIX C: UASB REACTOR PICTURES

Figure 9.1: Compositing tank during installation

Figure 9.2: Plumbing and compositing tank

Figure 9.3: Vault layout showing reactor

Figure 9.4: Portion of gas-solids-liquids separation device

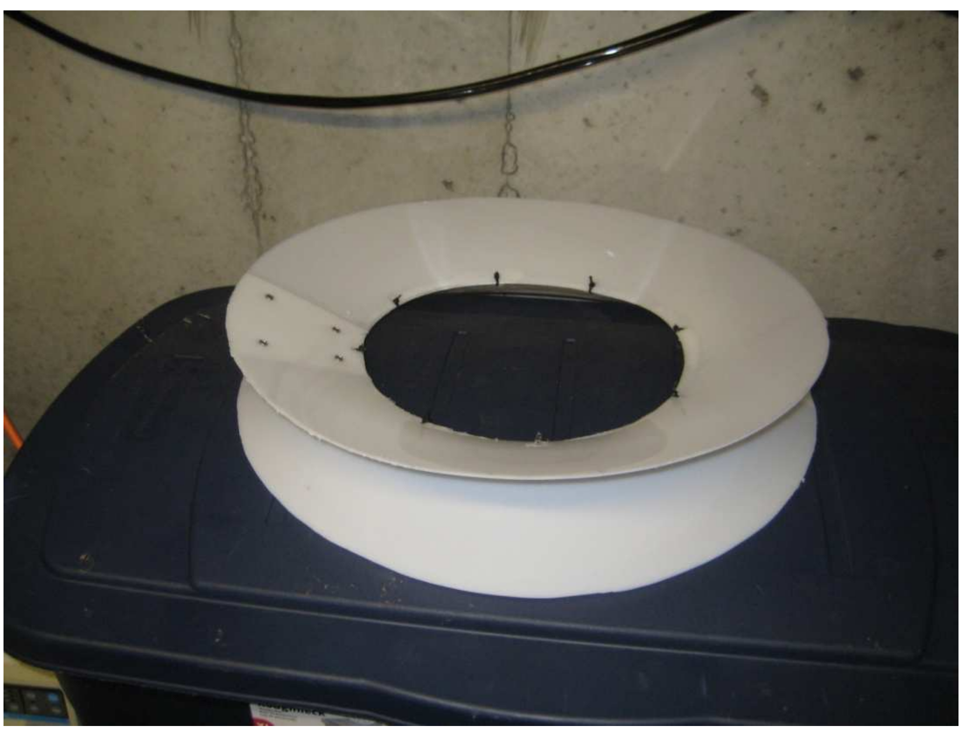


Figure 9.5: View of reactor looking down towards inlet

Figure 9.6: Reactor inlet prior to installation

Figure 9.7: Compositing tank mixer/shredder prior to installation

Figure 9.8: Compositing tank with mixer motor, DC drive, and peristaltic pump

Figure 9.9: Perforated PVC housing around compositing tank influent draw tubing

Figure 9.10: View of vault entrance, safety equipment, and foothills