THEORETICAL AND EXPERIMENTAL EVALUATION OF WASTEWATER RECIRCULATING BIOFILTERS

by

Zhifei Hu

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LIST OF SYMBOLS AND ABBREVIATIONS

$\phi_b(d)$	Fraction of volume filled by biomass at depth d
$\hat{q}$	Maximum specific rate of substrate utilization
$\gamma_{ut,2}$	Rate of substrate utilization in batch reactor period
$\gamma_{ut,1}$:	Rate of substrate utilization in the CSTR period
BOD_5	Five-day biochemical oxygen demand
COD	Chemical oxygen demand
CWQG	Canadian Water Quality Guideline
d	Depth in media, m.
d_{10}	Effective size
DF	Dosing frequency
DO	Dissolved oxygen
HAR	Hydraulic application rate
HLR	Hydraulic loading rate
ISF	Intermittent sand filter
K_s	Concentration giving one-half the maximum rate
K	Reaction rate coefficient
k_d	Distribution coefficient
L_{NH4}	NH_4^+-N loading rates
L_{org}	Organic loading rates
MCWWTP	Mill Cove Wastewater Treatment Plant
MDL	Method detection limit
NH_4^+-N	Ammonium nitrogen
NO_3^-	Nitrate nitrogen
O/M	Operation and maintenance
OLRs	Organic loading rates
OWTSs	Onsite wastewater treatment systems
Q_f,	Flow rates of filter effluent
Q_r	Flow rates of the recirculation tank effluent

Q_s	Flow rates of septic tank effluent
RBF	Recirculating biofilter
R_c	Organic removal rate achieved by crushed glass layer
RCGF	Recirculating crushed glass filter
R_d	Total organic removal rate of dual media RBFs
R_g	Organic removal rate achieved by geotextile layer
RGTF	Recirculating geotextile filter
RPF	Recirculating peat filter
RSF	Recirculating sand filter
RTF	Recirculating textile filter
S_1	Projected recirculation tank water quality;
S_2	Recirculation tank water quality at the end of a dose
S_3	Recirculation tank water quality at the beginning of a dose
SA_m	Surface area of the medium
SEM	Scanning electronic microscopy images
S_f	Component concentrations in the filter effluent
S_r	Component concentrations in the recirculation tank effluent
S_s	Component concentrations in the septic tank effluent
STE	Septic tank effluent
THM	Trihalomethanes
t_{max}	Maximum thickness of biofilm
TN	Total nitrogen
TP	Total phosphorous
TSS	Total suspended solid
UV	Ultraviolet
X	Concentration of active biomass

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ABSTRACT

Recirculating biofilters (RBFs) are an alternative system to traditional onsite wastewater treatment systems. This innovative design has come into wide use recently for small and rural communities where building properties and infiltration has limited the use of conventional septic systems. However, current design practice for recirculating biofiltration and the robustness of the current design is fairly limited in scope as it is based on "working knowledge". Thus, it is critical to develop an improved understanding of the biological and chemical processes for system trouble-shooting purposes and adapting this technology to non-traditional situations.

The overall objective of this thesis was to develop an improved understanding of recirculating biofilters that would lead to a more robust design approach for the sustainable implementation of these on-site systems. To obtain this objective, this study involved the use of bench-, pilot- and full-scale RBF systems in a series of controlled and uncontrolled experiments to analyze various impact factors on RBF performance. At the bench-scale, the factors investigated included hydraulic loading rate, dosing frequency, recycle ratio and filter media. The results showed that dosing frequency had a significant impact on RBF performance in terms of BOD_5 removal. Based on this conclusion, a target study compared five different dosing frequencies (including 48, 96, 144, 192 and 240 times per day) for municipal wastewater treatment. The results of the bench-scale experiments showed that a dosing frequency of 192 times per day could provide improved performance as compared to other dosing frequencies tested in this study. In addition, this research also conducted a series pilot-scale RBF experiments to examine the impact of ventilation locations on RBF performance. The experimental results showed that the RBF ventilated from both the top and the bottom sidewall of the filter bed could provide better performance than the RBFs ventilated from the only the top or the bottom sidewall of the filter beds. Four field-scale RBF systems were evaluated in this study to compare the long-term performance of RBFs treating multi-residential wastewater. Both bench- and field-scale studies found that crushed glass could be applied as an effective RBF medium. It was also found that geotextile could be another effective medium for RBFs. Although peat is widely applied in single-pass packed bed filters for domestic wastewater treatment, this study found that peat could not provide satisfactory performance as a RBF medium based on poorer removals of BOD_5 and ammonium (NH_4^+-N). Finally, this study investigated a dual-media RBF for treating municipal wastewater with geotextile and crushed glass as filter media. The filter beds were filled with these two types of media at five different volume ratios. Specifically, a crushed glass to geotextile volume ratio of 100:0, 75:25, 50:50 and 25:75 were evaluated at the bench-scale. The dual media RBFs were operated at both a low hydraulic loading rate (HLR) (i.e., 0.12 m3/m2/day) and a high (i.e., 0.40 m3/m2/day) HLR. The results showed that the dual media RBF with the geotextile volume percentage of 25 % could provide better performance than the other RBFs tested in this study under both the low and the high HLRs.

1. INTRODUCTION

Most onsite wastewater treatment systems utilize the combination of a septic tank and soil adsorption field to treat domestic wastewater. When properly constructed on adequate sites, these systems provide water that can be discharged into surface or ground water. However, these systems can fail due to poor construction practices which result in soil damage, or due to installation on unsuitable sites characterized by soil with low permeability, a high water table during part of the year or shallow rock depth. With increased interest in sustainable development for rural communities, large footprints for soil adsorption fields are becoming less desirable, and thus alternative technologies are more frequently being sought. Recirculating biofilters (RBFs) are an example of such an alternative treatment approach.

RBFs evolved from intermittent sand filters (ISFs) that were first introduced in the 1970s in Illinois, USA (Venhuizen, 2005). RBFs can be used for a broad range of applications including single-family residences, large commercial establishments and small communities. RBFs use sand, gravel, or other media to provide advanced secondary treatment of settled wastewater from septic tank effluents. They consist of a lined excavation or structure filled with uniform washed sand or other media that is placed over an under drain system. The wastewater is dosed at the surface of the filter bed through a distribution network and allowed to percolate down through the filter media to the underdrain system. The underdrain system collects and recycles the filter effluent to the recirculation tank for further processing or discharge. RBFs treat wastewater by physical means, such as straining and sedimentation, by chemical sorption, such as phosphorus removal, and by biological processes. The latter mechanism is the greatest strength of these treatment systems for on-site wastewater systems.

Although the use of recirculating biofilters, especially in onsite wastewater treatment systems, has increased steadily in recent years, the fundamental mechanisms for understanding and designing recirculating biofiltration is poorly described in literature.

Consequently, current design practice for recirculating biofilters and the robustness of said design is fairly limited in scope as it is based on "working knowledge" rather than an understanding of the fundamental mechanisms. Therefore, it is critical to develop an improved understanding of the biological and chemical processes of RBFs for trouble-shooting purposes and applying this technology to non-traditional situations.

Currently, RBFs are designed based on many impact factors, including filter media, hydraulic loading rates (HLRs), recycle ratio, dosing frequency, organic loading rates. Sand has been used as a recirculating biofilter medium in treating single family septic tank wastewater, and wastewater generated from small communities. Although sand is the most common filter medium for RBFs, recirculating sand filters (RSFs) can become relatively costly or difficult to implement when an adequate treatment medium cannot be found locally. Therefore, there is a need to introduce new types of filter media to increase the practicality of RBF implementation. In addition, it is of importance to investigate the impact of the interaction of these factors on RBF performance, such that this technology can be optimized and engineering design can be improved. By recognizing the gap between the current state knowledge and the demand for improved understanding of RBF systems, this study was conducted to achieve the following objectives.

1.1 Research Objectives

The overall objective of this thesis was to develop an improved understanding, through controlled laboratory experimentation and evaluations of a field-scale treatment plant, of recirculating biofilters that would lead to a more robust design approach for the sustainable implementation of this on-site wastewater treatment technology. Specifically, the following sub-objectives were studied to achieve the project's central goal:

i. To examine the short-term removal performance of biochemical oxygen demand (BOD_5), total nitrogen (TN), ammonium nitrogen (NH_4^+-N), total phosphorous (TP), fecal coliform, turbidity, and total suspended solids (TSS) in municipal wastewater in a prototype bench-scale RBF. This sub-objective was discussed through a step-by-step methodology with four phases:

- *Phase I* – 2^4 Factorial Analysis – This phase of the research involved a 2^4 factorial analysis to investigate the influence of four individual impact factors as well as the interactions among these factors on the performance of RBFs with BOD_5 removal as the responding parameter.
- *Phase II* – Post Factorial Analysis Investigation – This phase of the research involved an evaluation of the results of the 2^4 factorial analysis in Phase I to investigate the impact of the significant impact factor determined by the factorial analysis in Phase I on RBF performance.
- *Phase III* – Filter Media Comparison – This phase of the research focused on evaluating four types of RBF media. Specifically, silica sand, crushed glass, peat and geotexitle media were evaluated in terms of RBF treatment performance.
- *Phase IV* – Biomass Profile and Filter Depth Study – This phase of the research investigated biomass distribution within the filter bed using scanning electronic microscope (SEM) images. This phase also developed a laboratory controlled bench-scale RBF, which could be applied to simulate pilot- and field-scale RBFs.

ii. To compare three different ventilation locations in RBF design including at the top, the bottom sidewall, and both the top and the bottom sidewall of the filter beds. To achieve this sub-objective, two phases of study were conducted, including:

- *Phase I* – Observed Impacts of Ventilation Locations – This phase involved an experimental investigation of the impact of ventilation location on the performance of RBFs for the removal of organic compounds, nutrients and pathogenic microorganisms; and
- *Phase II* – Conceptual Model of Air Flow – This phase developed a conceptual model of air flow under different ventilation locations, based on recorded RBF performance in Phase I.

iii. To compare four different types of filter media, namely silica sand, crushed glass, peat, and geotextile in a field-scale study as well as to investigate the functions of the recirculation tank by the discussion of biological and chemical reactions and mass balance analysis for various water quality parameters.

iv. To examine the efficiency of a new media design (e.g., dual media) for RBFs treating domestic wastewater with crushed glass and geotextile as well as to evaluate the performance of these novel dual media RBFs by target analysis in the filter bed and the recirculation tank.

1.2 Organization of Thesis

The overall organization of this thesis is to report results for each phase of the research in the style of a referred journal paper. Specifically, chapters 4, 5, 6, and 7 are comprised of an abstract, introduction, materials and methods, results and discussion and conclusions that are specific to the research objectives. Each chapter is designed to meet the overall goal of understanding RBF design and operation.

Chapter 2 provides a literature review of RBF design and performance related to this investigation. It provides background information on traditional RBF design, application and performance for municipal and domestic wastewater treatment.

Chapter 3 consists of experimental materials and methods to describe the techniques that were used throughout the different phases of study. This chapter provides details of the physical set-up of the bench- and pilot-scale RBFs.

Chapter 4 presents the results of a series of experiments with bench-scale RBFs used to investigate different impact factors for the removal of BOD_5, TN, NH_4^+-N, TP, TSS, and fecal coliform. The impact factors investigated include filter media, hydraulic loading rates, dosing frequency, recycle ratio and filter depth.

Chapter 5 discusses the impact of three ventilation locations on RBF performance through pilot-scale studies. The three ventilation locations investigated included at the top, bottom sidewall, and both the top and the bottom sidewall of the filter bed. Based on the observed pilot-scale experimental results as well as data analysis, this chapter has developed a conceptual model of air flow through the RBF system under different ventilation locations.

Chapter 6 evaluates the long-term performance of four types of field-scale RBFs (including silica sand, crushed glass, geotextile and peat) conducted at the Municipality of Lunenburg, NS. This chapter focuses on the functions of the different components of field-scale RBFs by mass-balance analysis of various water quality parameters. The components discussed include the septic tank, the recirculation tank, the filter bed, and the UV reactor.

Chapter 7 examines the efficiency of a new media design (e.g., dual media) for municipal wastewater treatment with crushed glass and geotextile. This chapter mainly focuses on a comparison of observed performance of dual media RBFs with the geotexitle volume percentages of 0%, 25%, 50%, 75% and 100% in the bench-scale filter beds.

Chapter 8 presents the overall conclusions for the thesis and synthesizes key findings from this research.

Chapter 9 offers recommendations to the wastewater treatment industry and identifies several future research needs.

2. BACKGROUND

Onsite wastewater treatment systems (OWTSs) have evolved from the pit privies used widely throughout history to installations capable of producing a disinfected effluent that is fit for human consumptions (USEPA, 2002). In Canada, approximately 25-33% of the constructions are using onsite wastewater treatment system (Frank and Bright, 2001). About 18-20% of the populations rely on onsite wastewater treatment systems in Australia (O'Keefe, 2001). According to the U.S. Census Bureau (1999), approximately 23 percent of the estimated 115 million occupied homes in the United States are served by onsite systems, More than a third of the homes in the southeastern states depend on these systems, including approximately 48 percent in North Carolina and about 40 percent in both Kentucky and South Carolina. More than 60 million people in the United States depend on decentralized systems, including the residents of about one-third of new homes and more than half of all mobile homes nationwide (U.S. Census Bureau, 1999).

Water quality problems in rural portions of Canada are large and occur throughout the country. Contamination from easily identified sources such as septic systems in rural estate subdivisions and large farm operations is well recognized. In addition, contamination from small farms and isolated septic systems is also prevalent (Joy et al., 2001).

In the past, the approach to wastewater treatment in rural areas has been to rely on the large land area available for disposal and dispersal. When populations were small and densities low, this was sufficient to avoid large sale problems. However, the advent of agricultural intensification and relocation of many residents from the city to the country has led to progressively more serious problems in rural areas of Canada with respect of wastewater treatment. This was highlighted in rural well water surveys in Ontario (Rudolph and Goss 1993) in which over 25 % of the rural wells tested failed to meet drinking water standards. Indeed, in many cases of rural development (housing and agriculture), one of the key issues is that of on-site wastewater treatment. In response to this issue, may new technologies have been developed to handle on-site wastewater, from a variety of sources, in rural settings.

2.1 Traditional Onsite Wastewater Treatment Systems (OSWTSs)

The traditional onsite systems have consisted primarily of a septic tank and a soil absorption field, also known as a subsurface wastewater infiltration system, or SWIS (USEPA, 2002). In this thesis, such systems are referred to as *conventional systems*. The septic tank removes most settleable and floatable materials and functions as an anaerobic reactor that partially promotes organic compounds removals. Septic tank effluent (STE) containing pathogens and nutrients is traditionally discharged to soil, sand, or other media absorption fields for further polish treatment through chemical, physical, and biological processes. Conventional systems work well if they are installed in areas with appropriate soils and hydraulic capacities; designed to treat the incoming waste load to meet public health, ground water, and surface water performance standards; installed properly, and maintained to ensure long-term performance (USEPA, 2002). Properly designed, maintained, and used, these systems have been shown to provide excellent treatment with useable lives of 20 or more years (Joy et al. 2001). Conventional systems or SWIS were reported to achieved BOD, TN, TP, and fecal coliform removals of 90-98 % (Siegrist et al., 1986 and U. Wisconsin, 1978), 10-40 % (Reneau 1977, Sikora et al., 1976), 85-95 % (Sikora et al., 1976), and 2.0-4.0 log reductions (Gerba, 1975), respectively. Typical pollutants of concern in effluent from onsite wastewater treatment systems are listed in Table 2.1 (USEPA, 2002).

Table 2.1 Typical pollutants of concern in effluent from onsite wastewater treatment systems.

Pollutants	Public health or water resource impacts
Pathogens	Parasites, bacteria, and viruses can cause communicable diseases through direct or indirect body contact or ingestion of contaminated water or shellfish. Pathogens can be transported for significant distances in ground water or surface waters.
Nitrogen	Nitrogen is an aquatic plant nutrient that contributes to eutrophication and dissolved oxygen loss in surface water; especially in nitrogen limited lakes, estuaries, and costal embayment. Algae and aquatic weeds can contribute trihalomethanes (THM) precursors to the water column that might generate carcinogenic THMs in chlorinated drinking water. Excessive nitrate-nitrogen in drinking water can cause methemoglobinemia in infants and pregnancy complications.
Phosphorus	Phosphorus is an aquatic plant nutrient that can contribute to eutrophication of phosphorus-limited inland surface waters. High algal and aquatic plant production during eutrophication is often accompanied by increases in populations of decomposer bacteria and reduced dissolved oxygen levels for fish and other organisms.

With the increased development in rural places and small communities, conventional systems are not able to meet the stringent corresponding regulations due to several reasons as follow:

i. Space limits

When populations were small and densities low, SWIS was sufficient to avoid large-scale problems. However, the advent of agricultural intensification and relocation of many residents from the city to the country has led to progressively more serious problems in rural areas with respect to wastewater treatment (Joy et al. 2002). In Joplin, Montana, the population grew 16.5 % between 1982 and 1997, but the land base consumed by that increase grew by 40.6 % (Frederick, et al., 2004). Nova Scotia (Canada) Department of Environment and Labor (2001) issued an amended Environment Act requiring that the minimum lot size for septic tank soil adsorption system for a single dwelling generating 1000 L/ day domestic wastewater should be between 2700 and 9000 m^2, and the exact size is contingent upon local soil characteristics and topography.

Current regulations in Pennsylvania require 1.2 m of aerobic soil above bedrock or a seasonal high water table for a site to be defined as suitable for permit issuance (Hepner et al., 2001). Only about one third of the area in the Unites States has soils suited for conventional subsurface soil absorption fields. System densities in some areas exceed the capacity of even suitable soils to assimilate wastewater flows and retain and transform their contaminants. In addition, many systems are located too close to ground water or surface waters and others, particularly in rural areas with newly installed public water lines, are not designed to handle increasing wastewater flows (USEPA, 2002). In respect to this issue, many new technologies have been developed to handle on-site wastewater, from a variety of sources, in rural settings, including filtration process, wetland, lagoons, and some aerobic biological treatment.

ii. Stringent Regulations

According to Environment Canada, some of the federal discharge guidelines for wastewater at the point of discharge are as follows:

- Wastewater discharged to freshwater lakes and low-flow streams, rivers and estuaries, and open coastline should have BOD_5 and TSS values of less than 5, 20, and 30 mg/ L, respectively;
- The fecal coliform concentration should be less than 200 colony forming units (CFUs)/100 mL;
- The ammonium concentration should be less than 1.0 mg/ L;
- The nitrate concentration should be less than 10 mg/ L; and
- TP should be less than 1.0 mg/ L

However, some municipalities require more stringent discharge guideline. For example, at Municipality of Lunenburg, NS, both BOD and TSS at discharge point should be less than 20 mg/ L. These stringent local regulations and by-laws imposed some challenges to those existed SWIS, especially when they are not operated and maintained properly.

iii. Increasing Failure Rates

A number of systems relying on outdated and under-performing technologies (e.g., cesspools, drywells) still exist, and many of them are listed among failed systems. Moreover, about half of the occupied homes with onsite treatment systems are more than 30 years old (U.S. Census Bureau, 1997), and a significant number report system problems. A survey conducted by the U.S. Census Bureau (1997) estimated that 403,000 homes experienced septic system breakdowns within a 3-month period during 1997; 31,000 reported four or more breakdowns at the same home. Studies reviewed by USEPA cite failure rates ranging from 10 to 20 percent (USEPA, 2000). System failure surveys typically do not include systems that might be contaminating surface or ground water, a situation that often is detectable only through site level monitoring.

Comprehensive data to measure the true extent of septic system failure are not currently collected by any single organization. Most available data are the result of incidents that directly affect public health or are obtained from homeowners' applications for permits to replace or repair failing systems. The 20 percent failure rate from the Massachusetts time-of transfer inspection program is based on an inspection of each septic system prior to home sale, which is a comprehensive data collection effort. However, the Massachusetts program only identifies failures according to code and does not track ground water contamination that may result from onsite system failures.

In addition to failures due to age and hydraulic overloading, OWTSs can fail because of design, installation, and maintenance problems. Hydraulically functioning systems can create health and ecological risks when multiple treatment units are installed at densities that exceed the capacity of local soils to assimilate pollutant loads. System owners are not likely to repair or replace aging failing systems unless sewage backup, seepage pooling on lawns, or targeted monitoring that identifies health risks occurs. Because ground and surface water contamination by onsite systems has rarely been confirmed through targeted monitoring, total failure rates and onsite system impacts over time are likely to be significantly higher than historical statistics indicate. For example, the Chesapeake Bay Program found that 55 to 85 percent of the nitrogen entering an onsite system can be discharged into ground water (USEPA, 1993). A 1991 study concluded that conventional systems accounted for 74 percent of the nitrogen entering Buttermilk Bay in Massachusetts (USEPA, 1993).

In fact, many conventional system failures have been linked to operation and maintenance failures. Typical causes of failure include unpumped and sludge-filled tanks, which result in clogged absorption fields, and hydraulic overloading caused by increased occupancy and greater water use following the installation of new water lines to replace well and cisterns.

2.2 Alternative Treatment Approaches

The traditional septic system and leaching bed is becoming unable to handle many of today's design requirements for rural homes. Increased use of water, building on smaller pieces of property, and building in areas of limited infiltration capacity suggest that conventional systems do not have the capacity of sustaining in many of these more restrictive areas. Alternative technologies have been developed and are currently sought for OWTSs. These technologies include constructed wetland, packed-bed media filter, trickling filter, fixed-film activated sludge, rotating biological contactor, lagoon, activated sludge, fixed-film bioreactor, sequencing batch reactor, and some disinfection facilities (such as ultraviolet light). Table 1-3 shows detailed commonly applied system for OWTSs (USEPA, 2002).

An entire range of systems has been developed that rely on filtration as an additional step in the treatment process. The simplest of these is the installation of an effluent filter to the outlet of a standard septic tank, before the effluent leaves the tank and enters the adsorption bed (Crites and Tchobanoglous 1998). Manufacturers such as Zabel and Orenco have actively marketed these devices to the point where is some states of the United States they are mandatory on all new systems. Some Canadian jurisdictions are currently considering similar requirements (Joy et al. 2001).

A number of systems use filters as a more integral component of their system. The Waterloo Biofilter and Ecoflo® peat filter are both examples of these. Single pass sand filters have also been used for rural wastewater treatment. These systems work much like the Waterloo Biofilter and Ecoflo System, except that the filter media is specially graded sand. They also provide tertiary treatment effluent quality. However, they are not able to achieve high level of denitrification process (USEPA, 2002).

Multi-pass (recirculating) filters are similar to single-pass filters, with the exception that a portion of the treated effluent from the filter is returned to a recirculation tank where it is to dilute the effluent from the septic tank before being applied to the filter. Bio-slims from the growth of microorganisms develops as films on the sand particle surfaces. The microorganisms in the slimes absorb soluble and colloidal waste materials in the wastewater as it percolates over the sand surfaces. The absorbed materials are incorporated into a new cell mass or degraded under aerobic conditions to carbon dioxide and water. Single-pass filters are most frequently used for smaller applications and at sites where nitrogen removal is not required. Recirculating filters are used for both large and small flows and are frequently used where nitrogen removal is necessary. Recirculating sand filters frequently replace aerobic package plants in many parts of United States because of their high reliability and lower operation/maintenance (O/M) requirements (USEPA, 2002). In addition, Duncan et al. (1994) showed that highly treated effluent from a recirculating sand filter could reduce the soil depth required for biological and chemical renovation of wastewater.

Table 2. 2 Commonly used treatment processes and optional treatment methods

Treatment objective	Treatment process	Treatment methods
Suspended solids removal	Sedimentation	Septic tank
		Free water surface constructed wetland
		Vegetated submerged bed
	Filtration	Septic tank effluent screens
		Packed-bed media filters (incl. dosing systems) granular peat, textile
		Mechanical disk filters
		Soil infiltration
BOD and NH_4^+-N removal	Aerobic, suspended growth reactor	Extended aeration
		Fixed-film activated sludge
		Sequencing batch reactors (SBRs)
	Fixed-film aerobic bioreactor	Soil infiltration
		Packed bed media filters
		Trickling filter
		Fixed-film activated sludge
		Rotating biological contactors
	Lagoons	Facultative and aerobic lagoons
		Free water surface constructed wetlands
Nitrogen transformation	Biological	Activated sludge
	Nitrification	Fixed film bioreactor
	Denitrification	Submerged vegetated bed
		Free water surface constructed wetland
	Ion exchange	Cation exchange
		Anion exchange
Phosphorus removal	Physical/chemical	Infiltration by soil and other media
		Chemical flocculation and settling
		Iron-rich packed-bed media filter
	Biological	Sequencing batch reactor
Pathogen removal	Filtration, predation, inactivation	Soil filtration
		Packed bed media filters
		Granular
		Peat, textile
	Disinfection	Hypo-chlorite feed
		Ultraviolet light
Grease removal	Flotation	Grease trap
		Septic tank
	Adsorption	Mechanical skimmer
	Aerobic biological treatment	Aerobic biological treatment

Although the use of recirculating and filters, especially in onsite wastewater treatment system has increased steadily in recent years, the fundamental mechanisms for describing wastewater treatment on recirculating biofiltration is poorly described in the literature. Consequently current design practice for recirculating biofiltration and the robustness of the current design is fairly limited in scope as it is based on "working knowledge" rather than an understanding fundamental mechanism. It is critical to develop an improved understanding of the biological and chemical processes for trouble-shooting and for applying this technology to non-traditional situations. Since some other types of filter media have been applied as the replacement of traditional silica sand, this thesis used recirculating biofilter (RBFs) instead of recirculating sand filters (RSFs) to refer to this innovative technology. Therefore, this thesis mainly focused on developing an improved understanding for RBFs, such that it could provide more robust design tools for future applications.

2.3 Background of Recirculating Biofilters

2.3.1 Development and History of Use

The most commonly used biofiltration treatment units comprise the following basic elements: (1) a container with a liner for holding the medium; (2) an underdrain systems for removing the treated liquid; (3) the filtering medium; (4) a distribution and dosing system for applying the liquid to be treated onto the filtering medium; and (5) supporting appurtenances. Biofiltration process for OWTSs can be effectively divided into two categories, including intermittent biofilters (IBFs) and recirculating biofilters.

Intermittent sand filters (ISFs) have been used for both treatment of individual home wastewater and community wastewater for well over 100 years (Crites and Tchobanoglous, 1998). The first attempt to treat wastewater with sand and gravel filters can be traced back to Ealing and Chorley, England, in the later 1860s (Frankland, 1870). By the late 1870s the concept of ISF was put into practice in Massachusetts. The first community to construct an ISF system was Lenox, Massachusetts, in 1876 (Mancl and

Peeples, 1991). The use of sand filters waned until the 1940s, through the use of pilot-scale systems that were shallower and the sand size was coarser than was used in the Massachusetts systems. The sand sizes ranged from 0.10 to 0.26mm, and the bed depths were 0.46 to 0.76m (Grantham et al., 1949). In the 1970s and 1980s the use of ISF for individual homes increased, with the work done at the University of Wisconsin and the state of Oregon leading the way (Crites and Tchobanoglous, 1998). Figure 2.1 shows the detailed design of ISFs.

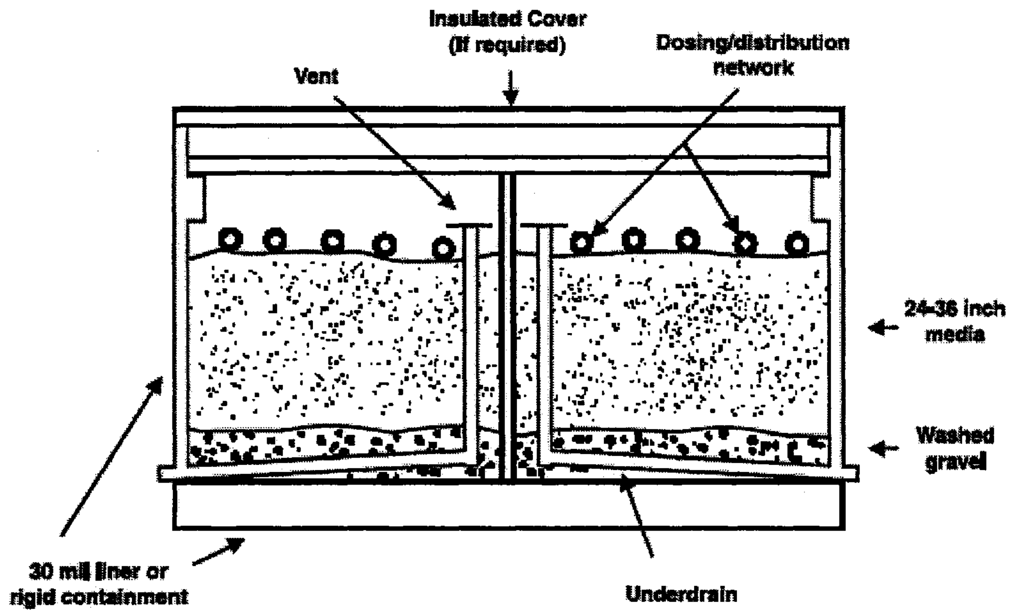


Figure 2. 1 Schematic flow diagram for ISFs (source: USEPA, 2002).

Multi-pass (recirculating) filters are similar to single-pass (intermittent) filters, with the exception that a portion of the treated effluent from the filter is returned to a recirculation tank where it is used to dilute the effluent from the septic tank before being applied to the filter. By diluting the strength of the septic tank effluent, higher application rates can be used. Recirculating biofilters evolved from ISFs in the 1970s by Hines and Farveau (1974). Recirculation was initially used to minimize odors when dosing open sand filters with septic effluent (Teske, 1979). It was immediately observed that recirculation also improved treatment efficiency for BOD and TSS removals. This was attributed to the more uniform loading schedule that recirculation can impart. In the mid-1970s RBFs systems were used to treat septic tank effluent prior to disposal in roadside ditches. In West Virginia, the sand medium was replaced with bottom ash, a hard, durable by-product from coal-fired power plants (Swanson and Dix, 1988). In Oregon, sand was replaced with fine gravel (Ronayne et al., 1984). The significance of these alternative media types is that surface clogging was greatly reduced or eliminated and higher loading rates were possible. A schematic flow diagram for a modern RBF is presented in Figure 2.2. As shown in Figure 2.2, the RBFs required the use of an additional tank (recirculation tank) as compared to the ISF.

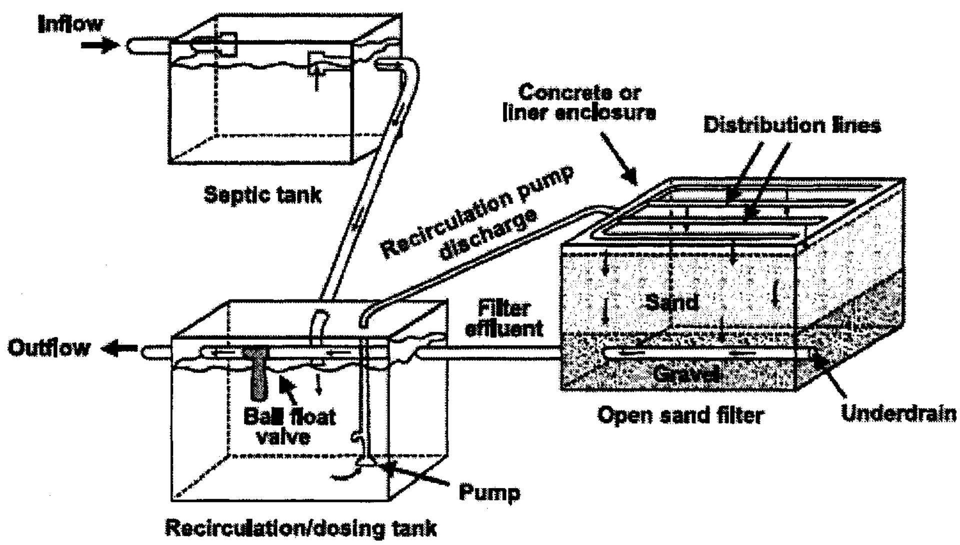


Figure 2. 2 Schematic flow diagram for a RBF (Source: USEPA, 2002).

2.3.2 Removal Mechanisms in RBFs

Shortly after filter is put into service, a thin bacterial film or biofilm begins to develop in the upper layers over the grains of the filter medium. The biofilm is of fundamental importance in the operation of the filter, because it retains, by means of absorption, the soluble and small colloidal matter and microorganisms found in settled wastewater. The retained material is decomposed and oxidized during the rest period between doses.

Soluble organic matter is utilized by the biofilm, while the absorbed colloidal material is solubilized enzymatically. The solubilized material is then transferred across the cell membrane and converted to end products. With each subsequent dose, some of the end products are transported farther into the bed, eventually being removed from the bottom of the filter. Larger particles are retained within the filter by means of filtration with mechanical straining and chance contact being the principal removal mechanisms. As with the soluble and colloidal matter, the coarse organic solids are also processed between doses, and in the early morning hours when the solids and organic loadings to the filter are reduced. As the filter mature, the film layer will develop throughout the filter. Because the larger solids are removed in the upper portion of the filter, the distribution of solids within the filter is nonlinear, with the largest accumulation occurring in the upper 10 to 20cm (Crites and Tchobanoglous, 1998).

When the flow is in a thin film, the oxidation of carbonaceous materials will occur in the upper portions of the filter bed. Simultaneously, ammonium will be converted to nitrate (nitrification). In turn, the nitrate will be converted to nitrogen gas (denitrification) in anoxic micro-sites within the filter if enough electron donor (such as BOD) and low oxygen concentration are maintained. Biological denitrification has been shown to occur under anoxic conditions and, for certain bacteria, under aerobic conditions (Robertson and Kuenen, 1990). The organisms responsible for denitrification utilize adsorbed carbon in particulate matter as the energy source. To sustain the performance of the filter, the microorganisms in the filter must be maintained in the endogenous growth rate (Crites and Tchobanoglous, 1998). If too much organic material is applied, the bacterial growth rate will increase and accumulation of material can occur, ultimately leading to failure.

Flow in a thin film is especially important if viruses are to be removed. In a recent study, it has been demonstrated that the formation of a uniform dense bacterial film, produced by increasing the number of doses per day, will have a significant effect on viruses removal (Emerick et al., 1997a).

2.3.3 Types of Applications

RBFs have been used to treat septic tank effluent from individual homes, clusters of homes, institutions, and small communities. They have also been used to nitrify pond effluent prior to discharge to constructed wetlands. They are frequently used to pre-treat wastewater prior to subsurface infiltration onsite where soil has insufficient unsaturated depth above ground water or bedrock to achieve adequate treatment. RBFs have also been used to treat septic tank effluent prior to UV disinfection and water reuse. RBFs are used for both large and small flows and are frequently used where nitrogen removal is necessary. RBFs frequently replace aerobic package plants in many parts of the country because of their high reliability and lower operation and maintenance (O/M) requirements (USEPA, 2002; Crites and Tchobanoglous, 1998).

Individual Systems. RBFs for individual dwelling septic tank effluent treatment have been widely reported in literature. Piluk and Peters (1994) reported that recirculating gravel filters were used extensively for individual family residences in Anne Arundel County, Maryland. In addition, recirculating sand filters (RSFs) have been successfully applied for single family septic tank effluent treatment in Michigan, Oregon, and Quebec as reported by Louden et al. (1985), Ronayne et al. (1982), and Roy and Dube (1994).

Small Community. RSFs were applied for treating average 15,000 gallon per day (gpd) of septic tank effluent in a small community, Wisconsin (Owen and Bobb, 1994). It achieved more than 90.0 % of BOD removals in winter operation. Another reported application for small community is at Stonehurst, which is a 47-lot subdivision located near the City of Martinez in Contra Costa County, California (Crites et al., 1997). The wastewater management system that was designed and constructed incorporates a number of innovative technologies, including watertight septic tanks with screened effluent filter vaults, high-head effluent pumps, a small-diameter variable-grade sewer, two pressure sewers, a recirculating granular medium filter, a UV disinfection unit, a subsurface drip irrigation systems for reuse of treated effluent, and a community of leachfield for wintertime disposal.

Restaurant. Recirculating pea gravel filter was used for treating a restaurant wastewater in Wisconsin (Ayres Assoc, 1998). The grease and oil influent and effluent for in this case were 119 mg/ L and 1 mg/ L, respectively.

Institution. A treatment facility consisted of a septic tank and a recirculating sand filter (RSF) was reported to treat wastewater from the two elementary schools and the middle school in Chaparral, New Mexico (Richardon et al., 2004). In order to enhance nitrogen removal and to comply with the 27mg/L effluent nitrogen requirements of the discharge permit, they applied simple process modifications to the RSF treatment facility. The process modification included installation of a pump and a recycle line to return the nitrified RSF effluent to the septic tank. With the improvement, the nitrogen removal efficiency observed at the plant increased to 54 percent.

2.3.4 Performance

Organic Matter Removal

USEPA (2002) reported that normally BOD and TSS effluent concentration of RBFs were less than 10 mg/ L when RBFs were treating residential wastewater. It was also concluded that the BOD was nearly completely removed if the wastewater retention time in filter media is sufficiently long for microorganisms to absorb waste constituents. Crites and Tchobanoglous (1998) reported that the removal of BOD in RBFs depends on the loading rate and the size of the filter medium. They concluded that RBFs could produce an effluent with less than 10 mg/ L BOD and TSS when loading rates are less than 0.20 m^3/m^2/day and the medium size is 3mm or less. Typical removals for BOD and TSS are presented in Table 2.3, which shows that the RBF effluent BOD_5 concentrations were typically less than 20 mg/ L even though the septic tank effluent BOD_5 was as high as 600 mg/ L (Ayres Assoc., 1998). It was also observed from Table 2.3 that most of recirculating sand filters were able to generate effluent with BOD_5 less than 10 mg/ L. Table 2.3 shows that 12 out of 14 RBFs generated effluents with TSS less than 10 mg/ L.

Nutrients Removal

Crites and Tchobanoglous (1998) reported that recirculating filters can produce a high quality, partially nitrified effluent. USEPA (2002) suggested that nitrifying microorganisms are able to thrive deeper in the surface layer where nitrification will readily occur. It is also suggested that nitrification occurs after the depletion of carbonaceous BOD in the wastewater. Nitrification tends to be complete, except in severely cold conditions (USEPA, 2002).

Nitrogen removal with recirculating filters is typically 40 to 50 percent (Crites and Tchobanoglous, 1998). They concluded that denitrification was determined by several parameters in RBFs, including anaerobic conditions, alkalinity, carbon source, and pH values. Compared with single-pass biofilters, RBFs have one more anaerobic component: recirculation tank which is designed for denitrification occurrence. USEPA (2002) reported that natural denitrification in recirculation tank could result in 40 to 60 percent removal of TN. While only 18 to 33 percent TN removal can be achieved by single-pass biofilters. This conclusion is quite consistent with the results presented by Crites and Tchobanoglous (1998). Literature has shown that TN removals by RBFs vary in a broad range. Sandy et al. (1988) and Elliott (2001) found 82 percent and 84 percent TN removals, respectively. However, only 41 percent and 43 percent TN removals were achieved by Loomis et al. (2001) and Christopherson et al. (2001), respectively.

Phosphorus (P) is a key issue in wastewater treatment due to the risk of eutrophication of the wastewater recipients. Few phosphorus removal processes are well developed for onsite wastewater systems application (USEPA, 2002). However Mckee (1998) reported that the removal of phosphorous by peat filters was very good, with most systems exceeding a 90 % removal. Effluent phosphorous concentrations were generally less than 0.8 mg/ L in most systems. However limited success has been reported because of inadequate operation and maintenance of mechanical equipment problems and excessive sludge production. Another notable method for high TP removals by RBFs has come with special filter materials that are naturally high in there concentration of some chemicals, including aluminum, iron, and calcium compounds. Some other studies of high-iron sands and high-aluminum muds indicate that 50 to 95 % of the phosphorus can be removed (USEPA, 2002). However, the life of these systems has yet to be determined, after which the filter media will have to be removed and replaced. Use of supplemental iron powder mixed with natural sands is also being researched (USEPA, 2002). All calcareous sands and other sand with high concentrations of these three elements will exhibit high phosphorus removal rates for some finite periods (USEPA, 2002).

Pathogenic Microorganisms Removal

USEPA (2002) reported that intermittent sand filter (ISF) or single pass sand filter could achieve fecal coliform removals of 2 to 4 log reductions. Cagle and Johnson (1994), Effert et al. (1985), Ronayne et al. (1982), Sievers (1998), Loomis et al. (2001), and Loomis et al. (2004) reported that ISF achieved fecal coliform removals of 3.0, 2.1, 2.8, 3.7, 3.5, and 3.9 log reductions, respectively. USEPA (2002) reported that RSFs could achieve 2 to 3 log reductions of fecal coliform. Table 2.3 showed that fecal coliform log reductions varied in a broad range, from 3.08-log (Reneau et al., 2001) to 0.90-log (Christopherson et al., 20001). Based on all of these results, Figure 2.3 showed that ISFs can generally achieve higher fecal ciliform log reductions than RSFs. This result was not consistent with data presented by Wren et al. (2004), who anticipated that total virus removal rates by RSFs with recycle ratio of 7 could be as high as 3.6-logs based on the fecal coliform removal by single pass filter ranging from 0.4 to 0.8 log reductions.

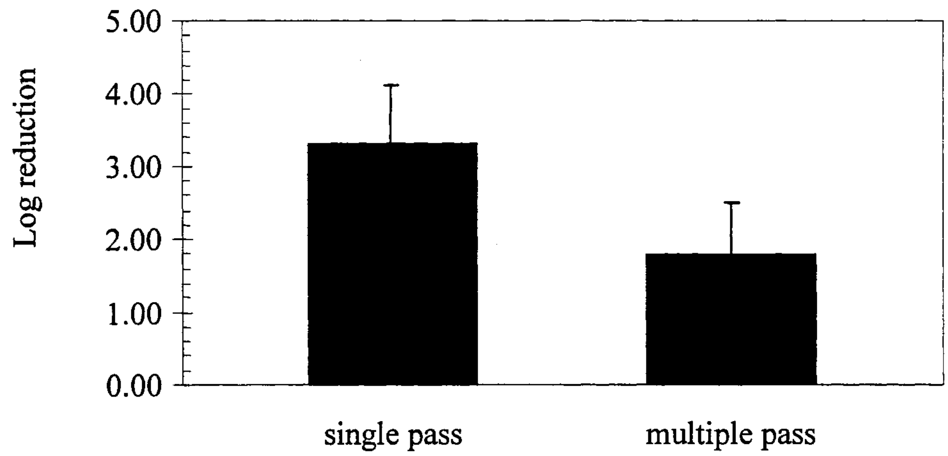


Figure 2.3 Comparison of fecal coliform removals (log reduction) by ISFs and RSFs (Error bars represent standard deviation from mean).

Table 2.3 RBFs Performance Summary.

Location	d_{10} (mm)	HLR $m^3/m^2/d$	BOD, mg/L		TSS, mg/L		TN		Fecal Ciliform (#/100mL)	
			Inf.*	Eff.**	Inf.*	Eff.	Inf.	Eff.	Inf.	Eff.
Michigan (Louden et al., 1985)	0.3 (sand)	0.04	150	6	42	6	55	26	$3.40x10^3$	$1.40x10^1$
Maryland (Piluk and Peters, 1994)	1.0 (sand)	0.14	235	5	75	8	57	20	$1.80x10^6$	$9.30x10^3$
Oregon (Ronayne et al., 1982)	1.2 (sand)	0.12	217	3	146	4	57.5	31.5	$2.60x10^5$	$8.50x10^3$
Quebec (Roy and Dube, 1994)	4.0 (sand)	0.94	101	6	77	3	37.7	20.1	$4.80x10^5$	$1.30x10^4$
Wisconsin (Ayres Assoc., 1998)	9.4 (pea gravel)	0.60	601	10	46	9	65.9	16	$>2.5x10^3$	$6.20x10^1$
Wisconsin (Owen and Bobb, 1994)	1.5 (sand)	0.11	80	8	36	6	NA	NA	NA	NA
Minesota (Christopherson et al., 2001)	1.2 (sand)	0.20	175	14	115	15	51	29	$4.90x10^5$	$6.10x10^4$
Pennsylvania (Hepner et al., 2001)	1.0 (sand)	0.20	77% removal		81% removal		46% removal		1.22-log reduction	
Virginia (Reneau et al., 2001)	1.2 (sand)	0.12	99	5.0	NA	NA	NA	NA	$2.65X10^4$	$2.20x10^1$
	1.2 (sand)	0.24	117	2	NA	NA	NA	NA	$1.00x10^5$	$1.45x10^2$
Rhode Island (Loomis et al., 2001)	1.9 (sand)	0.06	325	4	55	4	49	14	$2.7x10^4$	$2.7x10^3$
	Textile	0.76	325	19	55	13	49	29	NA	NA
New Mexico (Richardson et al., 2004)	Sand	NA	100	2	30	3	47	22	NA	NA
Rhode Island (Loomis et al., 2004)	Textile	0.09	300	4	200	4	125	70	$2.90x10^3$	$1.30x10^2$
		0.08	300	3	60	3	106	56	$3.9x10^4$	$1.30x10^3$
Kentucky (Byers et al., 2004)	9.4 (pea gravel)	N/A	215	12	75	7.3	NA	NA	$1.66x10^5$	$4.72x10^3$
New York (Elliott, 2001)	0.7 (crushed glass)	0.07	268	11	104	2.5	43.2	7	NA	NA

2.3.5 Design Considerations

Type and Size of Filter Media

The types of filter media used for RBFs are coarse sand, bottom ash, and fine gravel. RBFs must use a coarser media than single-pass filters because recirculation requires higher hydraulic loadings. Washed, graded sand has been the most common. The finest medium reported was the sand with an effective size of 0.3mm (Belick, 1986; and Louden et al., 1985). However, excessive clogging and short filter runs were observed with this sand size; and the investigators recommended an increased sand size of 0.6 mm. Most literature has shown that sand with d_{10} larger than 1.0mm is most common size in RBFs (Table 2.3). Although sand is the most common filter medium for RBFs, recirculating sand filters can become relatively costly or difficult to implement when an adequate treatment medium cannot be found locally (Roy et al., 1998). In this perspective, the development of a recirculating biofilter using another type of filter media offers an opportunity to connect the gap between filter media and media availability, as well as potentially improve technology performance.

Crushed glass is an alternative option to sand as a medium for RBFs (Darby et al., 1996; Emerick et al., 1997). It is an example of a medium that could provide opportunity to close the gap between filter media availability, as the total amount of Canadian recycled glass is typically high (e.g., greater than 60 %) (Firth 2002; Glassworks 1999). In Nova Scotia it was estimated that the return rate for beverage containers was over 83 %, which represents 216 million beverage containers (Firth, 2002). As a filter media for water treatment crushed glass was able to achieve low turbidity levels (<0.1 NTU), although particle counts were greater in crushed glass in comparison to sand (Rutledge and Gagnon, 2002). Crushed glass has been applied successfully as a RBF medium for domestic wastewater treatment in some places at New York State, USA (Elliott, 2001). In that study the recirculating crushed glass filters (RCGFs) removed TSS, BOD, NH_4^+-N approximately 98, 96, and 94 percent, respectively. Compared with traditional silica sand, crushed glass has several advantages, including i) crushed glass is less expensive than silica sand; ii) crushed glass is more environmentally friendly since it is recycled products; and iii) crushed glass can be pulverized into different sizes for specific design requirement, however sand has to be sieved for specific sizes.

Peat is partially decayed organic matter mainly of plant origin. It has been shown in numerous laboratory settings to effectively reduce several key components of wastewater. A laboratory study of peat columns was used to determine loading rates, allowable compaction, and nutrient removal parameters (Rock et al., 1984). A reduction of 96 % BOD, 80 % chemical oxygen demand (COD), and 93 % TSS was observed by Rock et al. (1984) in a peat column of 30cm with a bulk density of 0.12 Mg/ m^3. Little phosphorus reduction was observed and the authors concluded that NO_3 reduction was likely if anaerobic conditions persisted in the peat (David, 2004). Similar column and batch reactor studies indicated that as the thickness of peat increased treatment efficiency also increased (Rana and Viraraghaven, 1987; Viraraghaven and Rana, 1991). The previously described studies were performed in the laboratory, which were used to complement filed-scale studies and establish design parameters. Overall these studies all indicated the effectiveness of peat in reducing wastewater strength. Some full scale treatment systems utilizing peat have been used successfully to treat secondary effluent for many years (Farnham and Brown, 1972; Tilton and Kadlec, 1972; Guntenspergen et al., 1981; Nicholes and Boelter, 1982). Peat has been used as a filter medium for intermittent filters (USEPA, 2002); however, its use in recirculating biofilters has not been presented in the literature. In addition, the early performance of peat filter will produce an effluent with a low pH and a yellowish color contingent upon the type of peat used. This is accompanied by some excellent removal of organics and microbes, but would generally not be acceptable as surface discharge due to the low pH and visible color (USEPA, 2002).

The use of geotextiles for biological treatment has not been widely investigated, although its use for preventing biofouling in landfill leachate collection systems is widely reported (Yaman, 2003). Recently, textile was introduced as an innovative filter medium by some studies (Wren et al., 2004; and Loomis et al., 2004). These studies show that textile filters can offer a great advantage over natural media filters due to increased available surface area, high porosity, and lightweight compact design. In addition, Loomis et al. (2004) and Seigrist (2001) reported that textile filters generated effluent with BOD, TSS, TN, and fecal coliform removals of 99 %, 98 %, 44 %, and 3.4-log reductions, respectively. Textile filter is also advantageous for being dosed with much higher HLRs than recirculating sand filters. Loomis et al. (2001) found that a recirculating textile filter (RTFs) dosed with 0.76 m^3/m^2/day could provide removals for BOD and TSS of 99 % and 80 %, respectively. Christiane and Reid (1998) also reported that HLRs for RTFs could be as high as 1.00 to 1.80 m^3/m^2/day. They found that the surface area of textile filter for treating 100,000 L/ day of domestic wastewater was only 55-100 m^2, however the surface areas required for conventional leach-field and RSFs for treating same amount of domestic wastewater were 2500 m^2 and 500 m^2, respectively. Therefore, the characteristics of textile provide an interesting research gap that RTFs have potential to decrease the land area demand significantly by being operated under much higher HLRs than RSFs.

There are other types of filter media for RBFs. Screened bottom ash has been used in sizes ranging from 0.9 to 2.4 mm (Swanson and Dix, 19897; Crites and Tchobanoglous, 1998). Expanded shale appears to have been successful in some field trials in Maryland, but data are currently incomplete in relation to long-term durability of the medium (USEPA, 2002).

Hydraulic Loading Rate

Hydraulic loading rate is one of the design consideration parameters because it determines wastewater retention time in filter media where most of the organic matters can be biodegraded by biomass in the biofilm around particles under sufficient contact time. HLR for RBFs, based on peak flow, ranges from 0.12 to 0.24 m^3/m^2/day depending on the size of filtering medium (Crites and Tchobanoglous, 1998). They also reported that the typical HLR for recirculating filters, used to treat septic tank effluent from individual homes, is 0.20 m^3/m^2/day, based on forward peak flow from the septic tank. The typical design specifications for individual home RBFs by USEPA (2002) showed that HLRs varied based on filter media, including 0.12-0.20 m^3/m^2/day for sand filter, and 0.40 to 0.60 m^3/m^2/day for gravel filters. HLR of 0.94 m^3/m^2/day was applied for recirculating sand filters (RSFs) in Quebec (Roy and Dube, 1994). The sand with effective size (d_{10}) of 4mm was used in their study. The results showed that RSF generated effluent with BOD_5, TSS, TN, and fecal coliform of 6 mg/ L, 3 mg/ L, 20.1 mg/ L, and 1.30×10^4 CFU/ 100mL, respectively. Recirculating textile filters were successful for domestic wastewater treatment at Rhode Island with two levels of HLRs including 0.08 m^3/m^2/day and 0.76 m^3/m^2/day (Loomis et al., 2001; and Loomis et al., 2004). The results of these two studies showed that effluent BOD/TSS increased from 3/3 mg/ L to 19/13 mg/ L when HLRs increased from 0.08 to 0.76 m^3/m^2/day, respectively. However, both of these two RTFs generated effluent with BOD and TSS less than 20mg/ L.

Recirculation Ratio

Introduced by Hines and Favreau, recirculation was initially used to minimize odors when dosing open sand filters with septic tank effluent (Hines and Favreau, 1975; Teske, 1979). It was immediately observed that recirculation also improved treatment efficiency. This was due, in part, to the uniform loading schedule that recirculation can impart. The major factor, however, is dilution of the filter influent strength imparted by mixing forward flow with recirculation flow. The recirculation ratio increases the HLR to the filter without increasing the organic loading. Typical recirculation ratios range from 3:1 to 5:1 (USEPA, 2002). This conclusion was consistent with Crites and Tchobanoglous, (1998) that recirculation ratios were typically 4 or 5 to 1. USEPA (2002) also suggested that the recirculation ratio may need to increase to achieve the same level of treatment as the permeability of the media increases.

Dosing Frequency

It was found by a study at University of Florida at 1940' and 50' that, all other factors remaining equal, splitting the daily load into two doses increased removal efficiencies and allowed sand of a given size to be load more heavily (Venhuizen, 2005). This led to further investigation of more frequent loadings. USEPA (2002) reported that media characterization can limit the number of doses. According to that report, media reaeration must occur between doses. As the effective size of the media decreases, the time for drainage and reaeration of the media increases. For single pass filters, typical dosing frequencies are once per hour or less. Recirculating sand filters dose 2 to 3 time per hour or 48 to 72 times/ day.

Emerick et al. (1997) and Crites and Tchobanoglous (1998) introduced hydraulic application rate (HAR) as an appropriate term for the design of biofilters. HAR is defined as:

$$HAR, m/dose = \frac{Hydraulic \quad Loading \quad Rate \quad (m^3/m^2/d)}{Dose \quad Frequency \quad (doses/day)}$$

The importance of the HAR can be appreciated by reviewing the information presented in Table 2.4. At an HLR of 0.40 $m^3/m^2/d$, if the filter were dosed once per day, the volume of the dose would represent 217 percent of the field capacity of the medium. In these cases, most of the applied liquid will move down through and out of the filter. If, on the other hand, the filter were dosed 24 times/d the volume of the dose would represent about 9 percent of field capacity (Emerick et al. 1997). At 9 percent of the field capacity, the applied liquid will flow over the filter medium in a thin layer, maximizing the opportunity for removal. It is noteworthy that the field capacity of a filtering medium will increase as the film thickness within the filter increases.

Table 2. 4 Analysis of volume per dose for various hydraulic loading rates and dosing frequencies.

Hydraulic Loading Rate ($m^3/m^2/d$)	Dosing Frequency	Hydraulic application rate	Field capacity filled %
0.40	1	0.40	217
	2	0.20	107
	4	0.10	53
	8	0.5	26
	12	0.033	18
	24	0.0167	9
0.81	1	0.81	427
	2	0.40	217
	4	0.20	107
	8	0.10	53
	12	0.0675	26
	24	0.0338	18
0.163	1	0.163	855
	2	0.82	427
	4	0.41	217
	8	0.20	107
	12	0.14	71
	24	0.0679	36

Source: Crites Tchobanoglous, Small and Decentralized Wastewater Management Systems, 1998.

Depth of Filter Bed

Another basic design parameter is media depth. The studies conducted at University of Florida showed that most purification processes occurred within the top 23-30cm of the media (Furman et al., 1955; Calaway et al. 1952). However, Anderson et al. (1985) and USEPA (1980) asserted that additional media depth imparted consistency, assuring a more uniform effluent quality. The explanation for this conclusion was that deeper beds were not severely affected by rainfall and deeper beds allowed scrapping off of more filter media before replacement became necessary. Peeples et al. (1991) discussed filter depth in single-pass filters and found that the degree of nitrification that could be achieved with filter bed depth of 120cm reduced significantly with filter bed depth of 45cm, although BOD and TSS removals remained consistent in these two depths.

Crites and Tchobanoglous (1998) reported that typical filter bed depth for RBF ranges from 60 to 120 cm. They suggested that the bed depth beyond 60 cm were not usually warranted. It was asserted that the most typical depth of recirculating gravel filters was 60 cm (Crites and Tchobanoglous, 1998). In contrast, USEPA (2002) reported that most of the biochemical reactions occurred within the top 15 cm of the filter surface. USEPA (2002) also suggested that most of the carbonaceous BOD and ammonium nitrogen were removed as the wastewater percolated through this 15 cm active filter layer. It was also concluded that BOD was nearly completely removed if the wastewater retention was sufficiently enough for microorganisms to absorb the react with waste constituents. With the depletion of carbonaceous BOD in the percolating wastewater, nitrifying microorganisms were able to thrive deeper in this 15 cm active layer, where nitrification would readily occur. According to USEPA (2002) not only biodegradation of BOD and NH_4^+-N could be nearly completely depleted, most of the TSS were stained out at this active surface layer.

Organic Loading Rate

Organic loading rate became a concern because filter surface clogging can occur within the fine media filter in response to excessive organic loadings. As the biomass increases in the biofilter, it partially decreases the pore size and permeability by filling the pores of the surface layer of filter media. If organic loadings are too great, the biomass will increase to the point that surface layer gets clogged and is unable to provide further wastewater treatment. On the contrary, if the substrate supply in wastewater is less than the requirements for maintaining microorganisms growth, the microorganism will be forced into endogenous respiration which results in the death of cells. Consequently, the substrate, such as organic compounds and ammonium nitrogen, will not be removed efficiently (USEPA, 2002).

Unfortunately there were no sufficient data to establish a strong design guideline based on organic loading rates. Anderson et al. (1985) suggested that organic loading rates should be less than 0.03 kg/m^2/day in order to avoid premature clogging problem in sand filters. They concluded that the volume of sewage that could be purified wad depended on the organic matters contained in the sewage rather than the volume of wastewater in which these organic matters were held. Although it was well addressed that HLRs partially determined constituents removals in RBF, the robust understanding of the relationship between organic loadings and system performance had not yet been cleared defined (USEPA, 1980).

Typical values for organic loadings used in RBF varied from 0.01 to 0.04 kg/m^2/day (Crites and Tchobanoglous, 1998). However, USEPA (2002) suggested that BOD loadings on sand media should not exceed about 0.024 kg/m^2/day where effective size was approximately 1.0 mm and dosing frequency was 12 times per day. It was also asserted that organic loading rates should decrease with the decrease of effective size of the filter media. Some other studies have successfully applied higher organic loadings in RBFs in short term. But USEPA (2002) suggested that caution should be given to high organic loadings application due to the possible clogging problems. Gravel filter can be loaded with much heavier organic loadings, such as 0.10 kg/m^2/day, because of the larger pore size and greater permeability (USEPA, 2002).

2.3.6 Failure of RBFs

Both single- and multi-pass filters can fail when the physical (such as media size and type) hydraulic (such as HLRs and organic loading rates), and operational parameters (such as dosing frequency and recycle ratio) exceed certain limits. Crites and Tchobanoglous (1998) indicated that the typical failure mode was manifested by surface pounding between liquid applications. They suggested that failure always occurred by the accumulation of untreated organic/inorganic materials at the surface of filter media from the influent. Other typical failure mode was that there were too much fine particles present in the filter media from their study.

When the amount of substrates (or biodegradable compounds) exceeds the need for keeping the appropriate growth rate for microorganisms around the filter particles, an increased growth rate will occur. During the time until next dose, the organisms will process the stored organic matter, converting some of it into cell tissue. If there is no enough time for the organisms to reduce their mass through endogenous respiration between liquid applications (dose), the mass of organisms and unprocessed organic compounds will gradually accumulated and increased over time. As the biofilm thickness increases, the ratio of oxygen transfer, in turn, the biological activity is decreased, ultimately leading to the failure of the filter.

In a similar manner, if particulate organic matter and oil and grease are applied and trapped in the slim layer, the organisms must be able to solubilize and process the applied constituents before the next dose. If they are unable to do so, a gradual accumulation of solid will occur within the filter, which ultimately leads to failure of the biofilter.

2.4 Research Opportunities

The current state of knowledge of RBFs has evolved such that some practical experience has been gained through research and field testing. Unfortunately, much of the scientific understanding of the process functions and performance of RBF systems has been obtained based on individual 'rule of thumb' design and has not been fully and clearly demonstrated. Thus there is a recognized need for continued and expanded study of RBFs as an alternative option for wastewater treatment systems.

This thesis mainly investigated four research opportunities of RBFs, including: (i) optimal system design through controlled laboratory experimentation; (ii) extension development of system optimization through pilot-scale experimentation; (iii) demonstration and investigation of the performance of full-scale RBFs under actual field conditions; and (iv) development and evaluation of new dual media RBFs comprised of crushed glass and geotextile.

2.4.1 Impact Factors Investigation with Controlled Bench and Pilot-Scale RBFs

The previous sections of this chapter have discussed various design considerations for RBFs. Since most of these considerations have evolved individually from working experience, it is of great importance to investigate the impact of interactions among these factors on RBFs performance. These considerations are filter media, HLRs, recycle ratio and dosing frequency. Therefore, one of the research opportunities in this thesis was to investigate a set of factors and determine which factors produced an effect on RBF performance as well as estimate the magnitude of effects produced by changing the experimental factors. Bench-scale experiments were conducted in this thesis to evaluate these four individual parameters and their potential interactions due to the fact that bench-scale experiments can reduce time, cost and complexity in research as compared to field-scale experiments.

2.4.2 System study through pilot-scale experimentation

Pilot-scale experiments were conducted in this thesis as an extension of the system optimization study, since pilot-scale tests could investigate other emerging design considerations that bench-scale systems could not address. These studies focused on investigating variable filter depths and ventilation locations on RBF system performance. They are discussed in detail in Chapters 4 and 5.

2.4.3 Demonstration and investigation of the performance of full-scale RBFs under actual field conditions

Although bench-scale RBFs can be used to evaluate various design factors, RBFs were studied at the field-scale to provide verification of bench/pilot- scale RBFs evaluations and to address research questions of the long-term performance and reliability of this technology. Therefore, another research opportunity of this thesis was to investigate existing field-scale systems which could be viewed as providing experimental results that most representative true system performance. In addition, the performance of field-scale system can be used to verify the results of bench- and pilot-scale RBFs. The comparison between all of the RBF systems evaluated in this thesis can help to develop a more improved understanding of the treatment mechanisms in RBFs.

In addition, the literature has not shown the specific function of every component of a RBF system, such as the recirculation tank and the filter bed, as well as water quality variation in the different components of RBFs. However, there are challenges associated with investigating the functions of these components at the bench- and pilot-scale. Therefore, it was necessary to conduct field-scale tests to examine the water quality at every component of a RBF system, since optimization solutions could be generated if the reaction within every component could be thoroughly understood.

2.4.4 Development and evaluation of new dual media RBFs

Space limitations or treatment footprints have been previously identified as one of the design constraints for onsite wastewater treatment systems in rural communities. The design guideline indicates that the system's footprint, such as filter surface area, can be decreased by increasing HLRs for a given amount of daily flow of domestic wastewater (USEPA, 2002). Dual-media filters have been widely used in drinking water treatment processes (AWWA, 1999; Droste, 1997). In this type of design, fine media is placed under coarse media to provide pretreatment objectives, such as turbidity removal. Thus the fine media in the bottom layer is able to provide improved performance with less propensity for clogging occurrences. Therefore, the idea of investigating dual media for wastewater treatment provided a research opportunity to develop a novel RBF design for septic tank effluent treatment. Dual media RBFs are expected to provide satisfactory performance at high HLRs, such that the system footprint can be effectively decreased.

2.5 Conclusions

The background of onsite wastewater treatment systems were presented in this chapter. It was found that conventional systems are becoming increasingly unable to handle many emerging issues and design requirements for onsite wastewater treatment systems, including the space limitations, more stringent discharge regulations and increased system failures. Therefore, this chapter introduced alternative technologies that may provide a more sustainable design solution for onsite wastewater treatment . RBF technology was identified as being a viable design solution for onsite wastewater treatment due to economic and treatment performance considerations.

A comprehensive review of the development and history of RBF technology was presented in this chapter. Based on RBF removal mechanisms, types of application, treatment performance, design considerations and potential for system failure, this chapter introduced four research opportunities: (1) system investigation through bench-scale RBFs; (2) impact factors analysis through pilot-scale tests; (3) performance evaluation of field-scale RBFs and (4) development and evaluation of new dual media RBFs.

3. MATERIALS AND METHODS

3.1 Introduction

This chapter is intended to give a detailed description of the experimental systems, and the material and methods used for bench-, pilot- and field-scale experimental research. Additional materials and methods for specific experiments are supplied in subsequent chapters where appropriate.

3.2 Description of Bench-Scale RBF System

Bench-scale RBFs were operated at room temperature, approximately 20°C. Figure 3.1 shows a detailed design of bench-scale RBFs. The wastewater from the septic tank was pumped into the recirculating tank with velocity (v_1). Then the water was pumped with a velocity of v_2 into the filter (a glass column with length of 23 cm and diameter of 35 mm) from the recirculating tank. According to USEPA (2002), most of the biochemical reaction occurs within approximately 150 mm (6 inches) of the filter surface. Because of this a glass column with length 230 mm and diameter 35 mm was selected for this study to experimentally simulate the biological filtration process. The effluent from the filter was divided into two parts: the first part was pumped back into the recirculating tank with a velocity of v_3; the second part was discharged as the system effluent with a velocity of v_4. The wastewater was dosed into the filters at a constant frequency, which was controlled by a computer actuated pumping timing system. In this study, five different dosing frequencies were applied: 48, 96, 144, 192, and 240 times per day. In order to achieve these dosing frequencies, suitable on and off times for the pumps were chosen. For example, the pumps were on for 4 minutes and off for the next 11 minutes during a 15-minute cycle when the dosing frequency was 96 times per day.

Both the septic tank and the recirculating tank were sealed with covers to simulate the septic tank and the recirculation tank in field plants. The volumes of the septic and the recirculation tanks were adjusted based on the flow rate for each trial. For example, for a bench-scale RBF operated at a flow rate of 120 mL/day (or operated at a HLR of 0.12 $m^3/m^2/day$), the volumes of the septic tank and the recirculation tank were adjusted to be 120 and 180 mL, respectively. This adjustment is based on USEPA (2002) recommendation that the volume of the recirculation tank is normally 1.5 times of the volume of the septic tank.

In order to maintain aerobic conditions within the filter beds, ten 0.5 cm-diameter holes were drilled through the top cover of the column as shown in Figure 3.1. For the entire depth of the filter column, there was no sampling port. Therefore, only the water streams into and leaving the filters could be sampled as the influent to the filter bed and the effluent after biofiltration, respectively. In addition, water quality in the water stream into the filter bed is also the water quality of the recirculation tank effluent. Thus, this thesis also investigated the biochemical reactions in the recirculation tank. The filter media were supported by gravel at the bottom of the filter bed in order to avoid clogging problems with filter effluents. The filter columns were filled with sand, crushed glass, or peat without external pressure. Therefore, the porosity of the filter media was determined by the natural characteristics of these three types of filter media. When geotextile was used as the filter medium, a porosity of 0.90 was maintained by the following process:

i. Determine the volume filled by geotextile

$$\text{Geotextile volume} = \text{Column total volume} \times (1\text{-determined porosity}) \qquad [3.1]$$

ii. Determine the mass of geotextile

$$\text{Mass of geotextile} = \text{geotexitle volume} \times \text{geotextile density} \qquad [3.2]$$

iii. Fill column with determined mass of geotextile evenly

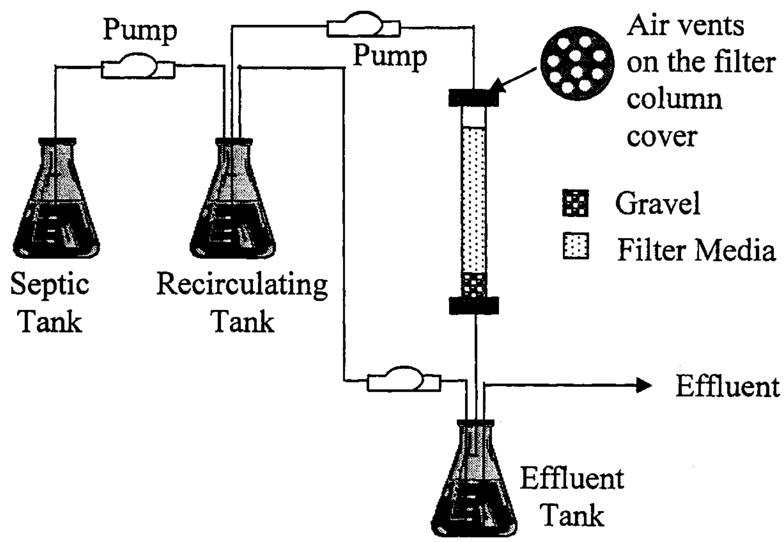


Figure 3.1 Physical set-up of controlled bench-scale recirculating biofilters.

The effective size (d_{10}) for both crushed glass and sand was 1.5 mm. The uniformity coefficient of crushed glass and sand were 1.44 and 1.89, respectively. Porosities of crushed glass and sand were 0.36 and 0.40, respectively. The Von post degree of decomposition of Peat was H-4. The pH value of peat was between 3.5 and 4.5, with organic matter content of more than 95%, nitrogen content of 0.5-1.0% of dry organic matter, and water content of 50-60%. The geotextile had a density of 95-100 kg/m^3, with specific surface area of 17,000 m^2/m^3, porosity of 90%, water holding content of less than 1%, and UV resistance at 1000 hours of 80%.

3.3 Description of Pilot-Scale RBFs Systems

Bench-scale RBFs were operated at room temperature, approximately 20°C. The physical set-up of pilot-scale RBFs was similar to Figure 3.1 except that the filter column was different. Figure 3.2 shows the detailed sketch of filter column used for pilot-scale RBFs. Pilot-scale RBFs were much more flexible in terms of filter bed depth than the bench-scale RBFs, since seven sampling ports existed over the entire depth of the column as shown in Figure 3.2. These sampling ports could be covered or uncovered during the system operation.

For example, the filter column could be filled up to N2, which provided 60cm of filter depth. For another instance, a 30 cm filter bed could be achieved by packing the filter column with 30cm of media. This would fall between sampling ports N4 and N5 (Figure 3.2). In addition, the pilot-scale filter column allowed the examination of the impact of ventilation locations on system performance. This could be investigated by ventilating filter beds at various locations, including the top, bottom sidewall, or both the top and bottom sidewall of filter bed, by opening the top sampling port (N1 and N2), bottom sampling port (N6 and N7) , or four of them together (N1, N2, N6, and N7), respectively. Clean and dry crushed glass was used as the filter medium for the pilot-scale RBF.

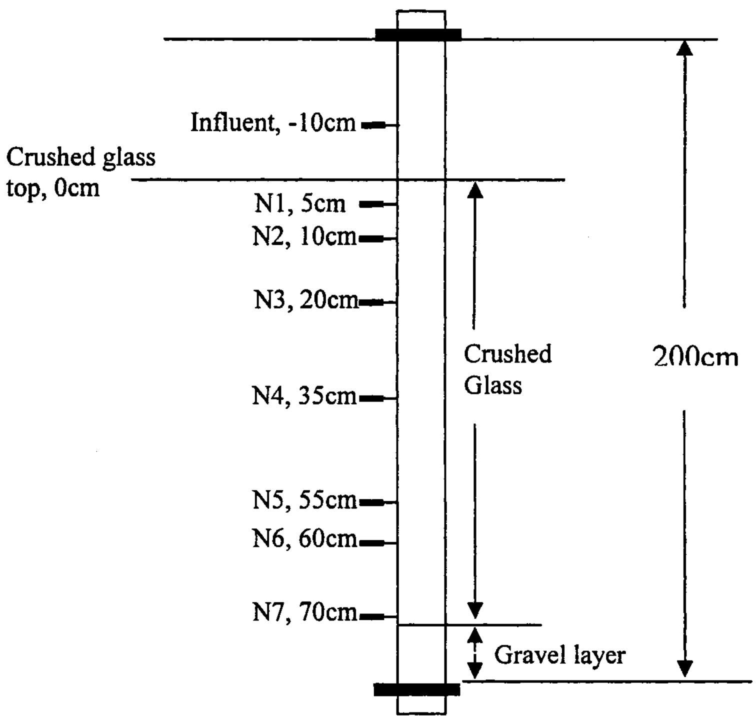


Figure 3. 2 Filter columns in pilot-scale RBFs.

3.4 Raw Water Characteristics for Bench- and Pilot-Scale RBFs

Raw wastewater from Mill Cove Wastewater Treatment Plant (MCWWTP) (Bedford, NS, Canada) was fed in the bench-scale experiments in this study. MCWWTP was constructed in 1969 and updated in 1997 to serve the communities of Bedford-Sackville, Bedford, NS, Canada, with capacity of daily flow of 7.5 million gallons, and peak flows of 16.5 million gallon per day. The process consists of flow equalization, primary clarification, pure oxygen activated sludge biological treatment, secondary clarification and ultra violet light disinfection. The plant meets and exceeds the effluent requirements of CWQG of 30 mg/ L BOD_5 and 30 mg/ L TSS. The influent wastewater from MCWWTP has an average initial BOD_5 of 170 mg/ L, TN of 50.0 mg/ L, NH_4^+-N 35 mg/ L, and Fecal Coliform (F.C) of 3.8×10^5 CFU/ 100maL. However, the raw water quality varied significantly over time during this study, and it is individually reported in the next chapters for all bench- and pilot-scale RBFs.

3.5 Water Balance and Loading Rates Determination in Bench- and Pilot-Scale RBFs

3.5.1 Water Balance

The flow rate for the RBFs was determined by:

$$Q = HLR \times Area \qquad [3.3]$$

Where Q is the flow rate of septic tank effluent, mL/ day; and Area is the filter column cross area, m^2.

The velocity profile in the bench- and pilot-scale system was determined by:

Septic tank effluent velocity, m^3/m^2/day: $v_1 = Designed \quad HLR$ [3.4]

Recirculating tank effluent velocity, m^3/m^2/day: $v_2 = (recycle \quad ratio + 1) \times v_1$ [3.5]

Recycled filter effluent velocity, m^3/m^2/day: $v_3 = (recycle \quad ratio) \times v_1$ [3.6]

RBF effluent velocity, m^3/m^2/day: $v_4 = v_1$ [3.7]

- Determine the volume per dose into the filter bed from the recirculation tank, mL/ dose:

$$Q_{dose} = (Q \times \text{recycle ratio})/\text{dosing frequency} \qquad [3.8]$$

3.5.2 Substrate Loading Rates

- Determine the organic loading rate, kg/m^2/day:

According to Leverenz et al. (2002), the organic loading rate is a measure of the oxygen demanding compounds (soluble and particulate organic materials) applied to the biofilter on an area basis. The OLR is calculated by multiplying the 5 day biochemical oxygen demand (BOD_5) or chemical oxygen demand (COD) by the HLR and an appropriate unit conversion factor. Therefore, the organic loading rate in this study was determined based on Equation 3.9:

$$L_{org} = HLR\ (m^3/m^2/day) \times C_{BOD}\ (mg/L)\ /\ (1000\ mg/L \times 1 m^3/kg) \qquad [3.9]$$

- Determine the ammonium nitrogen loading rate, kg/m^2/day:

$$L_{NH4} = HLR\ (m^3/m^2/day) \times C_{NH4}\ (mg/L)\ /\ (1000\ mg/L \times 1 m^3/kg) \qquad [3.10]$$

3.6 Sample Collection and Analysis

3.6.1 Chemical Samples

Five-day biochemical oxygen demand (BOD_5), total nitrogen (TN), ammonium nitrogen (NH_4^+-N), nitrate (NO_3^-), total phosphorous (TP), pH, total suspended solids (TSS), and turbidity were measured for collected samples.

Five-Day Biochemical Oxygen Demand

Five-day biochemical oxygen demand (BOD_5) was measured at Dalhousie University using standard technique as described in *Standard Methods for the Examination of Water and Wastewater (19th edition).* The incubation bottles (300 mL) were filled with sample to overflowing and sealed to ensure air tightness of the bottle. These bottles were incubated at room temperature (20°C) for 5 days. Dissolved oxygen (DO) was measured before and after incubation, and the BOD_5 was computed using the difference between initial and final DO. A BOD_5 buffer was made using the recipe as described in *Standard Methods for the Examination of Water and Wastewater (19th edition).* Since this study mainly focused on unchlorinated municipal and domestic wastewater, the water samples contained satisfactory microbial populations. Therefore, the seeding process was not necessary in this study. The dilutions used in this study were as follow: 2% for septic tank effluent, 5% for recirculating tank effluent, and 10% for RBFs effluent.

The method detection limit (MDL) of BOD_5 measurement was determined to be 2 mg/ L in this study. Values measured less than 2 mg/ L were reported as 2 mg/ L for purposes of data analysis and interpretation in the thesis.

Total Suspended Solids (TSS)

Due to the limit volume of bench- and pilot-scale RBFs, only field scale RBF effluents TSS were reported in this thesis. TSS of the field-scale RBF effluent was measured at Dalhousie University using standard technique (2540.D) as described in *Standard Methods for the Examination of Water and Wastewater (19^{th} edition)*. A well-mixed sample was filtered through a weighed standard glass-fiber filter and the residue retained on the filter was dried to a constant weight at 103 to 105 °C. The increase in weight of the filter represents the total suspended solids.

Total Nitrogen (TN)

Total nitrogen (TN) was measured at Dalhousie University TN was measured with method 10071, HACH DR/2500. An alkaline persulfate digestion converts all forms of nitrogen to nitrate. Sodium metabisulfite is added after the digestion to eliminate halogen oxide interferences. Nitrate then reacts with chromotropic acid under strongly acidic conditions to form a yellow complex with an absorbance maximum at 410 nm.

Ammonium Nitrogen (NH_4^+-N)

Ammonium nitrogen (NH_4^+-N) was measured at Dalhousie University using Method 8038, *HACH DR/2500 Spectrophotometer*. The Mineral Stabilizer complexes hardness in the sample. The Polyvinyl Alcohol Dispersing Agent aids the color formation in the reaction of Nessler Reagent with ammonium ions. A yellow color is formed proportional to the ammonia concentration. Test results are measured at 425 nm.

Nitrate Nitrogen (NO_3^-)

Nitrate nitrogen (NO_3^-) was measured at Dalhousie University using Method 8039 *HACH DR/2000 Spectrophotometer*. Cadmium metal reduces nitrates in the sample to nitrite. The nitrite ion reacts in an acidic medium with sulfanilic acid to form an intermediate diazonium salt. The salt couples with gentisic acid to form an amber colored solution. Test results are measured at 500 nm.

Total Phosphorus (TP)

Total phosphorus (TP) was measured at Dalhousie University using Method 8190, *HACH DR/4000 Spectrophotometer*. Phosphates present in organic and condensed inorganic forms (meta-, pyro-, orother polyphosphates) must be converted to reactive orthophosphate before analysis. Pretreatment of the sample with acid and heat ad described in the procedure provides the conditions for hydrolysis of the condensed inorganic forms. Organic phosphates are converted to orthophosphates by heating with acid and persulfate. Orthophosphate reacts with molybdate in an acid medium to produce a mixed phosphate/molybdate complex. Ascorbic acid then reduces the complex, giving an intense molybdenum blue color. Test results are measured at 880 nm.

Turbidity , Temperature and pH

Due to the limit volume of bench- and pilot-scale RBFs, turbidity instead of TSS was recorded as a measurement of the clarity of water for the effluents from the recirculation tank and the filter bed. Although it is predominately used for potable water, it is also occasionally used to assess the performance wastewater treatment processes. Turbidity is an indirect measurement of the amount of suspended solids in the water.

Turbidity, temperature and pH were measured on a regular basis using an HACH RATIO Turbidimeter (Hach Company, Loveland, CO) and Orion model 230A pH meter, respectively.

3.6.2 Microbial Samples

Water samples for suspended microbial analysis were collected from the septic tank effluent, recirculating tank effluent, and the filter effluent. Water samples were collected three times a week during the first three weeks and five times a week during the fourth week and fifth/sixth week if it was necessary. Samples were collected in pre-cleaned 100 or 200-mL glass vials as appropriate. All the samples were stored at 4°C before they were measured. Fecal coliform tests and scanning electronic microscopy images (SEM) were taken of the biofilm around the filter media.

Fecal Coliform

Fecal coliform was measured using Standard Method 9222.D, as described in *Standard Methods for the Examination of Water and Wastewater (19th edition).* Sample volumes for filtration were determined in order to yield counts between 20 and 60 fecal coliform colonies per membrane. The mFC Agar from EM Science® was used for the detection of fecal coliforms by the membrane filtration technique. The volumes of raw septic tank effluent used were always 0.1, 0.01, 0.001 mL in order to obtain the most valid colonies. The volumes of RBF treated effluent used were 10, 1, 0.1 mL. The samples were incubated at a temperature of 45.0 ± 0.2°C for 24 hours after the filtered samples had been transferred into the fecal coliform agar plates.

Scanning Electronic Microscopy Images (SEM)

The surface structure of the biofilm around the filter media in the RBFs was determined by examining scanning electronic microscope (SEM) images with Hitachi 2700 at Dalhousie University. Samples were taken from various depths within the RBFs, including top, middle, and bottom. The filter media were sampled and dried at room temperature for 24 hours before SEM images were taken.

3.6.3 Statistical Analysis

Results from the experiments were analyzed using factorial analysis. Factorial analysis was used to discuss which factors impacted RBFs performance significantly, such that further experiments setup could be determined. In this thesis, an analysis of variance (ANOVA) was used to conduct the factorial analysis. In addition, single factor ANOVA test and paired t-test were used to statistically analyze and compare observed experimental results in some chapters of this thesis. Unless otherwise noted, the level of significance (α) for those tests was 5%. For analysis purpose, all of the experimental factors were treated as qualitative variables.

3.6.4 Sampling Frequency

Water samples for chemical analysis were collected from the effluent of the system as well as the recirculation and septic tanks. Samples were collected three times a week during the first three weeks and five times a week during the fourth week and the fifth/sixth week if it was necessary (i.e, the system did not achieve a pseudo-steady-state condition within five weeks). Samples were collected in pre-cleaned 100 or 200-mL glass vials as appropriate. All the samples were stored at a temperature of 4 °C after they were taken.

3.6.5 Data Point Employed for Analysis

Only the data points during the pseudo-steady-state period were employed for systems performance evaluation and comparison. Due to the large amount of measured water quality data, the numbers of data points employed for analysis were not reported in body text. However, these data points were reported and highlighted with an asterisk in appendices. Generally six to ten data points were applied for system performance evaluation.

3.6.6 Experimental Duplication

During the whole study period, there were 16 silica sand RBFs were conducted under laboratory controlled conditions. Another 27 crushed glass RBFs were also conducted under laboratory controlled conditions through bench- or pilot-scale tests. In addition, three bench-scale geotextile RBFs were investigated during the study period. This study also operated 10 bench-scale RBFs for dual media RBFs study. Laboratory controlled bench- and pilot-scale RBFs were generally operated four to seven weeks.

Field-scale RBFs study were conducted between June and August, 2004, and the treatment plant started to provide service at December, 2003. During the study period, there were 15, 10, 5, and 12 filtration samples measured for sand, crushed glass, peat, and geotextile RBFs, respectively. In addition, the septic tank, the recirculation tank, and the UV reactor each had more than 20 samples measured during the study period.

4. FACTORS AFFECTING RECIRCULATING BIOFILTERS FOR TREATING MUNICIPAL WASTEWATER

A version of this chapter has been accepted for publication by the *Journal of Environmental Engineering and Science*

4.1 Abstract

The main objective of this investigation was to understand the impact of different impact parameters as well as interactions of these factors on treatment efficacy of recirculating biofilters (RBFs) under the controlled bench- and pilot-scale experimental conditions. The impact factors investigated were filter media, HLRs, dosing frequency and recycle ratio. Laboratory controlled experiments demonstrated that dosing frequency impacted treatment performance significantly based on a 2^4 factorial analysis. As an alternative biofiltration media, crushed glass was found to perform similarly to silica sand in terms of BOD_5 removal. This chapter also compared four types of filter media, including sand, crushed glass, peat, and geotextile. The results showed that the peat was not an effective medium for RBFs, since the peat filter provided the lowest ammonium (NH_4^+-N) (84.5 %) removal with an effluent NH_4^+-N of 5.5 ± 2.3 mg/ L. Geotextile was found to be an effective medium for RBFs. Scanning electronic microscope (SEM) images of the biofilm formed on the media at different depths of the filters suggested that filter depth should be considered as a design criterion as well, since biofilm development did not distribute evenly over the entire filter bed depth. Finally, this chapter found that a short-term study of a 15 cm bench-scale RBF could be applied to simulate and predict the performance of pilot-scale or field-scale RBFs.

4.2 Introduction

As previously summarized in Chapter 2, RBF performance is determined by many impact factors. These impact factors include filter media, hydraulic loading rates (HLRs), dosing frequency (DF), recycle ratio, organic loading rates (OLRs), and other parameters. Although some of these impact factors have been investigated individually by other RBFs studies, the impact on the performance of RBFs from potential interactions of these factors has not been comprehensively addressed. Therefore, the main objective of this chapter was to gain a more comprehensive understanding of the impact of these different parameters on RBF treatment performance as well as investigate potential interactions of these parameters on RBF treatment efficacy under controlled bench- and pilot-scale experimental conditions.

4.3 Methodology

This chapter follows a step-by-step methodology to achieve the main goal. In particularly, four steps, named Phase I, II, III, and IV in this chapter, were conducted as follows:

- **Phase I**: **2^4 Factorial Analysis** – This phase of the research involved a 2^4 factorial analysis to investigate the influence of four individual impact factors as well as the interactions among them on RBFs performance with BOD_5 removal as the responding parameter. The four impact factors used in the factorial analysis were filter media (silica sand and crushed glass), hydraulic loading rates (0.12 and 0.20 $m^3/m^2/d$), dosing frequency (48 and 96 times per day), and recycle ratio (2:1 and 4:1). The factorial analysis in this phase was conducted using experimental results from 16 bench-scale RBFs.
- **Phase II**: **Post Factorial Analysis Investigation** – This phase of the research involved an evaluation of the results of the 2^4 factorial analysis in Phase I to determine the significant impact factor. Another set of bench-scale RBFs was conducted in this phase.
- **Phase III**: **Filter Media Comparison** – This phase of the research focused on evaluating four types of RBFs media. Specifically, silica sand, crushed glass, peat and geotexitle media were evaluated in terms of RBF treatment performance based on RBF influent and effluent water quality parameters (i.e., BOD_5, TN, NH_4^+-N, turbidity and fecal coliform). In this phase, another four bench-scale RBFs filled with four types of bench-scale RBFs were conducted.
- **Phase IV**: **Biomass Profile and Filter Depth Study** – This phase of the research investigated biomass distribution within the filter bed by examining the surface density of biofilm around the filter media at different filter depths using scanning electronic microscope (SEM) images. In addition, this phase conducted another laboratory controlled 30 cm pilot-scale crushed glass RBF. The performance was compared with one of the 15 cm bench-scale RBFs

conducted in Phase III to examine the effective filter depth for biological and chemical reactions, including BOD_5 and NH_4^+-N removals within filter beds.

The bench- and pilot-scale RBFs experimental setups have been described in detail in Chapter 3. Additional information is supplied at every research phase of this Chapter to clarify the methods used. Throughout the experiments, samples were collected from septic tank and system effluents and analyzed for BOD_5, TN, NH_4^+-N, pH, fecal coliform, turbidity and TP using methods described in detail in Chapter 3.

4.4 Phase I – A 2^4 Factorial Analysis

4.4.1 Experimental Design

In this phase, 16 bench-scale RBFs were operated under controlled laboratory conditions. Table 4.1 presents the values of the design parameters used in these 16 RBFs.

Table 4.1 Summary of experimental setup for the 2^4 factorial analysis.

Trial	Media	Dosing Frequency (times per day)	HLR ($m^3.m^{-2}.d^{-1}$)	Recycle Raito	Organic loading rates ($kg.m^{-2}.d^{-1}$)
1	Sand	96	0.12	4:1	0.020
2	Sand	96	0.12	2:1	0.020
3	Sand	96	0.20	4:1	0.034
4	Sand	96	0.20	2:1	0.034
5	Sand	48	0.12	4:1	0.020
6	Sand	48	0.12	2:1	0.020
7	Sand	48	0.20	4:1	0.034
8	Sand	48	0.20	2:1	0.034
9	Crushed glass	96	0.12	4:1	0.020
10	Crushed glass	96	0.12	2:1	0.020
11	Crushed glass	96	0.20	4:1	0.034
12	Crushed glass	96	0.20	2:1	0.034
13	Crushed glass	48	0.12	4:1	0.020
14	Crushed glass	48	0.12	2:1	0.020
15	Crushed glass	48	0.20	4:1	0.034
16	Crushed glass	48	0.20	2:1	0.034

The filter beds were filled with two types of media, including silica sand and crushed glass, up to 15 cm. The wastewater was dosed into the filter beds with a dosing frequency of 48 and 96 times per day. When dosing frequency was 48 times per day, every dose lasted for eight minutes and the average volume per dose were 2.4 mL and 4.0 mL for HLRs of 0.12 and 0.20 $m^3/m^2/d$, respectively. When dosing frequency was 96 times per day, every dose lasted for four minutes and the average volume per dose were 1.2 mL and 2.0 mL for HLRs of 0.12 and 0.20 $m^3/m^2/d$, respectively. Wastewater from the bench-scale septic tanks were pumped into the bench-scale recirculating tanks at HLRs of 0.12 and 0.20 $m^3/m^2/d$. Two recycle ratios were evaluated, specifically 4:1 and 2:1.

4.4.2 Results and Discussion

Bench-Scale RBFs Development

Samples were collected with the frequency described in Chapter 3 from the septic tank and RBF effluent throughout the bench-scale trials. The crushed glass and sand RBFs achieved a pseudo-steady-state operating condition after four weeks, as the effluent BOD_5 concentration was essentially constant after four weeks of operation. Figure 4.1 shows the effluent BOD_5 concentrations of the crushed glass RBF during six weeks operation time (Trial 10). The sand filter had a similar trend as the crushed glass filter for BOD_5 removal as shown in Appendix: A – Biofiltration – BOD_5.

The raw water BOD_5 from the septic tank varied from 150 mg/ L (minimum) to 382 mg/ L (maximum) as shown in Figure 4.1. Regardless of the highly variable influent BOD_5 concentration, the effluent BOD_5 concentration from the surface ventilation RBF was within a smaller range from 8 mg/ L (minimum) to 22 mg/ L (maximum). These results demonstrate the treatment robustness of RBFs as they were able to achieve a very stable effluent quality throughout the course of the experiments. The remaining discussion of the Phase I experimental results are based on the system performance after the pseudo-steady-state was achieved. In particular, Appendix A shows the start data point (marked with *) employed for system performance evaluation and comparison.

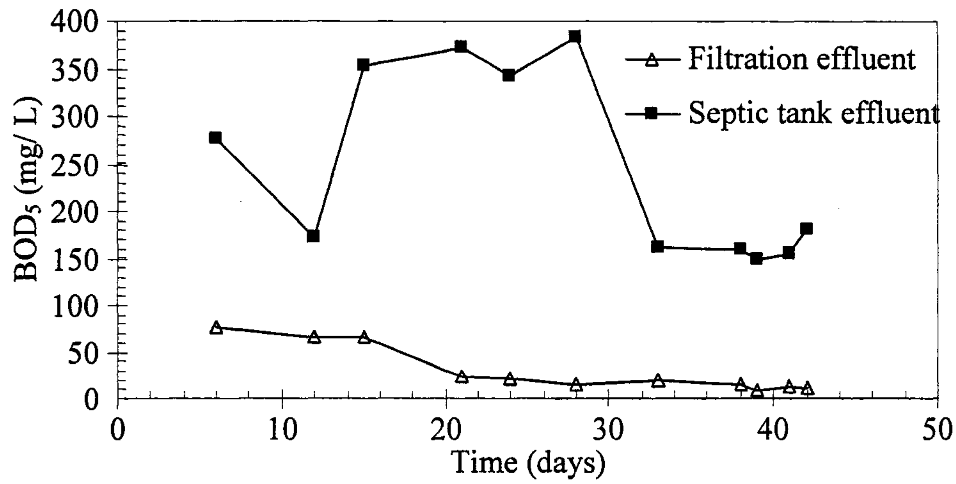


Figure 4.1 Crushed glass filter development over time (Trial 10).

RBF Performance

BOD_5 Removal. As presented in Figure 4.2., both the bench-scale recirculating sand filters (RSFs) and the recirculating crushed glass filters (RCGFs) provided BOD_5 removal greater than 85 %. In particular, the average BOD_5 removal was higher at a dosing frequency of 96 times per day (97 and 93 %, for the RSF and RCGF respectively) than at a dosing frequency of 48 times per day (89 and 89 %, for the RSF and RCGF, respectively). The Canadian Water Quality Guideline (CWQG) for BOD_5 of wastewater effluents is less than 5, 20 and 30 mg/ L for effluent discharges to freshwater lakes and low flow streams, rivers and estuaries, and open coastline as receiving bodies, respectively. The experimental results presented in Figure 4.2 demonstrate that the effluent BOD_5 concentration was less than 20 mg/ L for all of the treatment conditions evaluated in this study. These results suggest that RSF and RCGF design are capable of achieving the regulatory requirements for BOD_5 of effluent discharges.

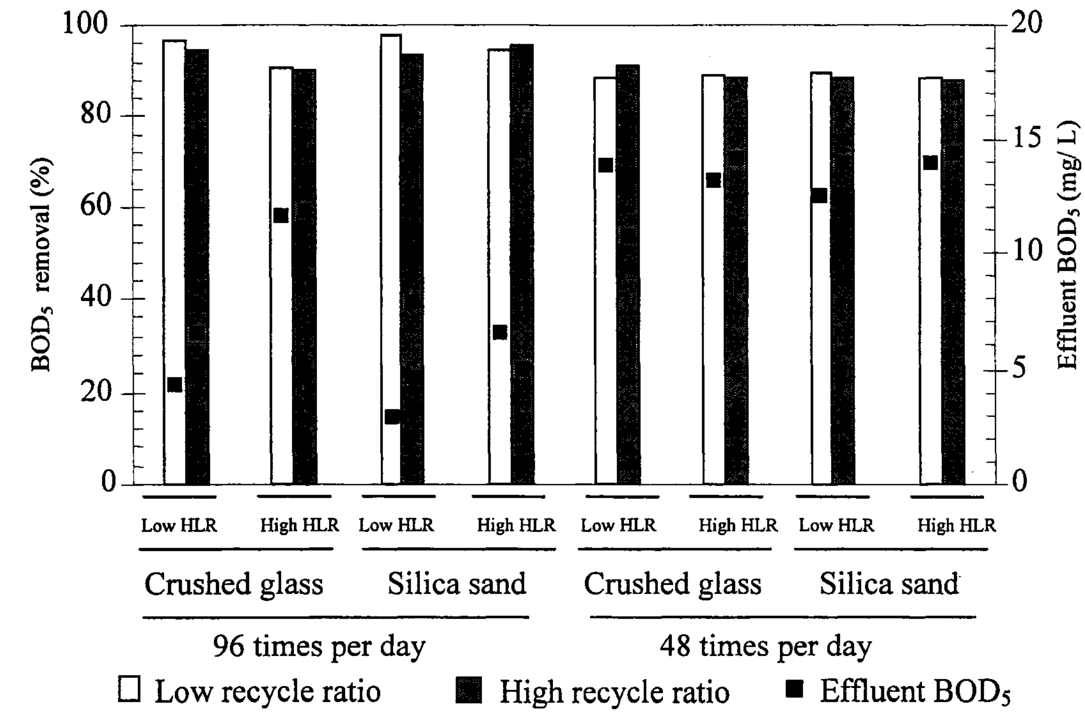


Figure 4.2 BOD_5 removals and effluent concentration for bench-scale experiments involving crushed glass and sand.

Figure 4.2 also demonstrates that the RBFs dosed with a lower HLR (i.e., 0.12 m^3/m^2/day) provided a slightly higher BOD_5 removal than the RBFs dosed with a higher HLR (i.e., 0.20 m^3/m^2/day). However, the results of this study found no apparent trend relating the impact of filter media and recycle ratio on BOD_5 removal. The impacts of HLR and recycle ratio are further discussed in more detail in Section 4.4.2.3.

NH_4^+-N Removal. Removal of NH_4^+-N in the bench-scale RSFs and RCGFs ranged from 92.7% to 99%, with an average effluent concentration of NH_4^+-N for all 16 trials of less than 1.0 mg/ L. These results are consistent with the results of full-scale trials reported by the USEPA (2002) that found ammonium nitrogen tended to be removed completely. The detailed recorded data for NH_4^+-N concentrations in samples collected in this study are presented in Appendix: A – Biofiltration – NH_4^+-N.

TN Removal. The percent removal of TN in the bench-scale RSFs and RCGFs varied greatly from 13.2 to 77.3 % as shown in Figure 4.3. USEPA (2002) reported that TN removals in RSFs are between 40 and 60 %. Sandy et al. (1988) and Elliott (2001b) reported 82 and 84 % TN removals from a RSF and RCGF, respectively. CWC (1979) reported a TN removal of 29 % from a crushed glass biofilter treating residential wastewater in King Co. WA. Jantrina et al. (1998) reported 36 % TN reduction from a multi-pass gravel biofilter for treating residential wastewater in Gloucester, MA. Although the results of the current study indicate that the TN removal in RSFs is more variable than TN removal reported by the USEPA (2002), the results are consistent with other reported TN removals reported by others.

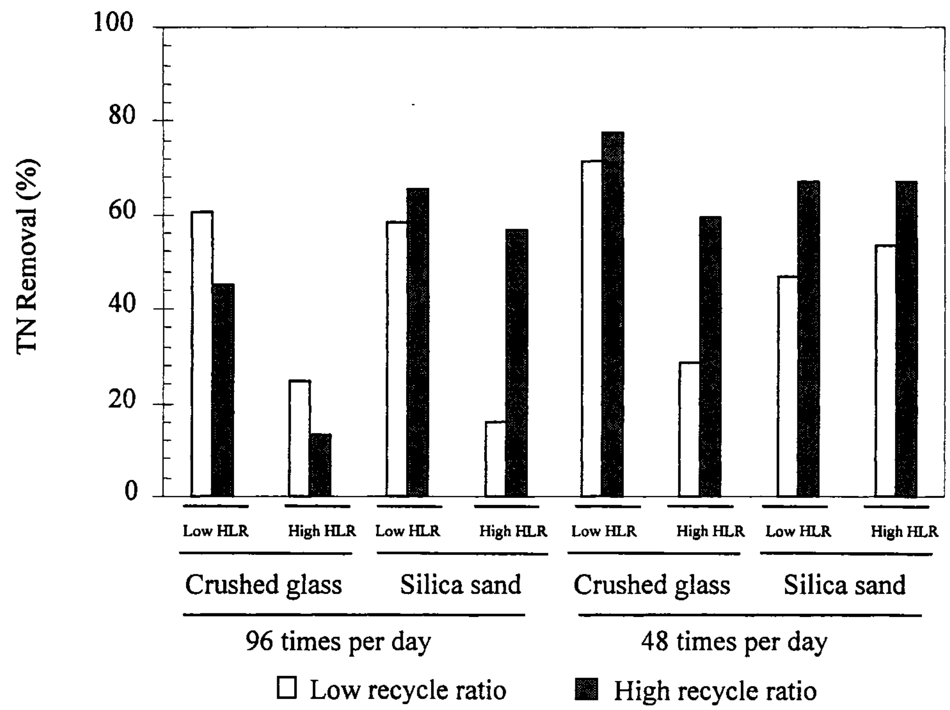


Figure 4.3 TN removal in bench-scale RSFs and RCGFs

Fecal Coliform Removal. Fecal coliform in the filter effluent was tested as an indicator of the RBFs to remove pathogens. Most of the effluent samples tested showed fecal coliform counts higher than 200 CFU/ 100 mL as shown in Figure 4.4, the requirement of CWQG at the point of discharge. The results of the current study found fecal coliform log reductions ranged from 1.5 to 3.6, which is higher than log reductions reported by the USEPA (2002) (i.e., 2.0 to 3.0 log removal).

CWC (1979) studied two field-scale crushed glass biofilters used for residential wastewater treatment. The results of that study found average effluent fecal coliform counts of 1,600 and 2,300 CFU/ 100 mL for the systems at King CO. WA and Ronald WA., respectively. Christopherson et al. (2001) reported effluent fecal coliform counts of 110,000 CFU/ 100 mL from a gravel recirculating biofilter used for residential wastewater treatment at Minnesota. The fecal coliform counts reported in the CWC (1979) and Christopherson et al (2001) studies are considerably higher than those reported in the present study. The low effluent fecal coliform counts found in the present study is probably related to the lower fecal coliform counts in the raw septic tank effluent (municipal wastewater) as compared to typical coliform counts found in residential wastewaters. However, Loudon et al. (1985) observed an effluent fecal coliform count of 14 CFU/ 100 mL from a gravel recirculating biofilter treating residential wastewater in East Lansing, MI. Based on the results of the present study as well as results reported in literature, it can be concluded that RBF effluent fecal coliform counts can be highly variable. Consequently a disinfection process would be required to ensure pathogen inactivation in the RBFs effluents is achieved to comply with wastewater discharge regulations.

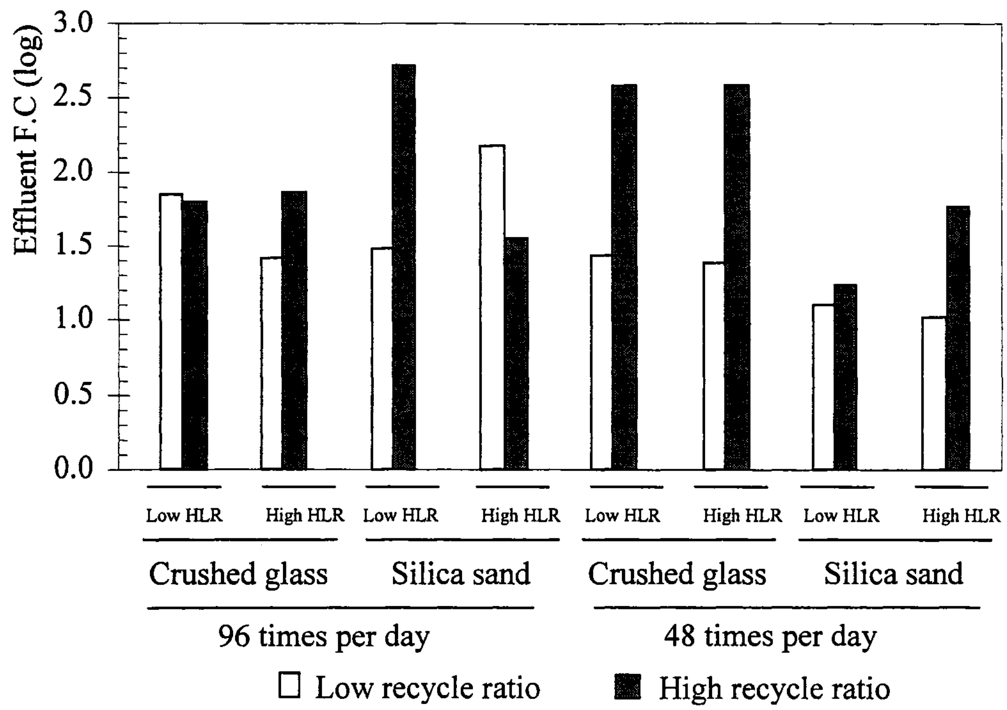


Figure 4.4 Fecal coliform counts in bench-scale RSFs and RCGFs

Factorial Analysis Results

Four experimental factors were examined in Phase I, including filter media (silica sand and crushed glass), HLRs (0.12 and 0.20 $m^3/m^2/d$), recycle ratio (2:1 and 4:1), and dosing frequency (48 and 96 times per day). Based on an ANOVA analysis shown in Appendix H, A factorial analysis with BOD_5 removal as the responding parameter showed that the dosing frequency significantly impacted the BOD_5 removal (α =0.05). This analysis demonstrated that the other experimental factors did not significantly impact BOD_5 removal (α =0.05). The detailed calculations for the factorial analysis are provided in Appendix H: Factorial Analysis.

Dosing Frequency. Furman et al. (1955) reported that more frequent dosing frequency allows higher loading rates to be readily accommodated with no degradation in treatment and without premature clogging of the filter. Crites and Tchobanoglous (1998) also concluded that high dosing frequency can improve BOD_5 removal in RBFs. These results are consistent with those of the current study. For same amount of treated wastewater, high dosing frequency can reduce the volume of individual doses. Small dose volumes are preferred because the flow through the porous media will occur under unsaturated conditions with higher moisture tensions (USEPA 2002). Better wastewater media contact and longer residence times occur under these conditions.

Crites and Tchobanoglous (1998) used hydraulic application rate (HAR) to assess the flow mode of RBFs under different dosing frequencies and HLRs. Chapter 2 introduced the concept of HAR for single-pass packed bed filters. HAR, a parameter used to describe filter bed loading (Leverenz et al., 2000), is defined in Equation 4.1 for multi-pass packed bed filters:

$$HAR = \frac{\mathrm{Re}cycle \quad ratio \times HLR}{do\sin g \quad frequency} \qquad [4.1]$$

The HAR is a measure of the depth of water introduced to the bed with each dose (Leverenz et al. 2000). The intent of introducing HAR is to match the dosing frequency to the field capacity, the moisture that is retained in the filter upon free drainage. For example, if one dose per day is used with an HLR of 0.04 $m^3/m^2/d$, the volume of wastewater per dose is over 200 % of the field capacity of the media. On the other hand, if 24 doses per day were used with the same HLR, then the volume of each dose would represent only 9 % of the field capacity (Yaman, 2003). Lower field capacity applied in each dose indicates that the liquid will flow through the filter medium in a thin layer, increasing substrate sorption and oxygen transfer in the pores of the filter medium. However, smaller doses do not draw as much air into the filter bed in each cycle, or produce the anaerobic condition required to provide denitrification (Yaman, 2003). Hence, the selected dosing interval depends upon the effluent standards.

Crites and Tchobanoglous (1998) also provided a conceptual model to describe the different flow modes within the filter bed under different dosing frequencies, as shown in Figure 4.5. This model suggests that saturated and unsaturated conditions are determined by the volume of liquid applied in every dose. It is clear in the conceptual model shown in Figure 4.5 that a high dosing frequency can result in a small volume every dose, which results in an unsaturated flow mode. When dosing frequency is low, saturated or partial saturated flow mode can exist in RBFs as described in Figure 4.5.

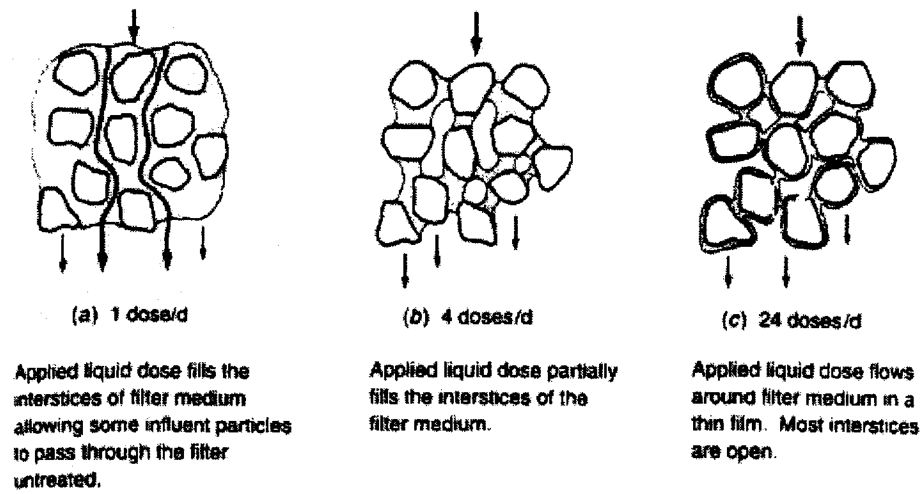


Figure 4.5 Effect of dosing frequency for single pass packed bed filters (Adapted from Crites and Tchobanoglous, 1998).

Filter Media. Previous RBF studies conducted by Darby et al. (1996) and Emerick et al. (1997a) indicated that crushed glass can perform similarly to sand that has similar media size and uniformity. The Town of Oswego, New York, has used crushed glass as a filter medium and has been in compliance with environmental regulations (Elliott, 2001a). The results from Phase I of the current research also showed that crushed glass can be an effective medium for RBFs. Furthermore, the results of a full-scale study showed that pulverized glass with a grain size distribution similar to the ASTM C-33 standard for fine aggregate was an effective substitute for the traditional silica sand used in filters treating effluent from residential septic tanks (The Clean Washington Center, 1995).

Crushed glass can be produced from pulverizing recycled glass. Of the estimated 940,000 tons (850,000 Mg) of container glass generated per year in the Canadian market place, slightly less than half was collected by recycling programs. In Nova Scotia, it is estimated that the return rate for beverage containers is more than 83%, or roughly 216 million beverage containers per year.

By demonstrating that crushed glass and sand performed similarly in the bench-scale RBFs trials of Phase 1, the results support the idea that crushed glass can be used as an alternative media to traditional silica sand in recirculating filters. These finding opens the possibility that a new industrial market for recycled glass could be developed, a market that could assist efforts to improve the performance of onsite wastewater treatment systems in areas with limited supplies of sand.

Compared with traditional silica sand, crushed glass has several advantages, including i) crushed glass is less expensive than silica sand; ii) crushed glass is more environmentally friendly since it is a recycled product; and iii) crushed glass can be pulverized into different sizes for specific design requirements, however sand has to be sieved for specific sizes.

Hydraulic Loading Rates. Factorial analysis found that the HLRs of 0.12 and 0.20 $m^3/m^2/d$ did not significantly impact the treatment efficacy of the biofilters based on the bench-scale RBF trials of Phase 1. This finding is of practical importance, because a HLR of 0.20 $m^3/m^2/d$ would require less footprint of the RBF than a HLR of 0.12 $m^3/m^2/d$.

The BOD_5 removal under different HLRs has been well reported. Table 4.2 presents the results of other studies that have evaluated BOD_5 removal under variable HLRs between 0.12 and 0.20 $m^3/m^2/d$. It is observed from Table 4.2 that there is no clear trend for BOD_5 removals at different HLRs. It can be concluded that BOD_5 removal higher than 93 % can be achieved in RBFs under HLRs between 0.12 and 0.20 $m^3/m^2/d$. The present study found that BOD_5 removal could be less than 90% when RBFs were dosed 48 times per day. This observed low BOD_5 removal in the present study is not consistent with literature reported results as shown in Table 4.2. However, the BOD_5 removal reported in Table 4.2 cannot be analyzed with the consideration of dosing frequency due to the lack of reported dosing frequency applied in these studies.

Table 4.2 Summary of BOD_5 removal under different HLRs

HLR ($m^3/m^2/d$)	BOD_5 Removal (%)	Reference
0.10	95	Leverenz et al., 2002
0.12	96	Critis et al., 1997
0.12	98	Jantrania et al., 1998
0.13	96	Louden et al., 1985
0.14	98	Venhuizen et al., 1998
0.14	99	Venhuizen et al., 1998
0.15	96	Venhuizen et al., 1998
0.20	93	Christopherson et al., 2001
0.20	96	Venhuizen et al., 1998
0.23	98	Reneau et al., 1998

Recirculation Ratio. Recirculation was initially used to minimize odors when dosing open sand filters with septic tank effluent (Hines and Favreau 1975; Teske 1979). It was immediately observed that recirculation also improved treatment efficiency. However, a major design factor is the dilution of the filter influent strength can be decreased by mixing forward flow with recirculation flow. Several recirculation ratios have been reported, such as 3:1 to 5:1 (USEPA 2002), and 7:1 (Piluk 1988). However, the factorial analysis conducted in Phase I of the present study showed that different recycle ratios did not impact RBFs performances significantly in terms of BOD_5 removal.

CWC (1997) reported on the performance of a crushed glass RBF used for treating residential wastewater at King Co. WA. The recycle ratio and the BOD_5 removal in that study were 1:1 and 96 %, respectively. Elliot (2001b) reported 94 % BOD_5 removal for a crushed glass RBF for residential wastewater treatment in Oswego NY with a recycle ratio of 3:1. Christopherson et al. (2001) reported 93 % BOD_5 removal for a gravel RBF treating residential wastewater in Minnesota with a recycle ratio of 5:1. These literature results demonstrate that there is no clear trend between BOD_5 removal and recycle ratios in RBFs. This conclusion is consistent with the factorial analysis results found in the present study.

4.4.3 Phase I Summary

In summary, the results of **Phase I** demonstrate the following:

1. Alternative filter media

    - Crushed glass should be more widely applied as an environmentally friendly medium for RBFs.

2. Improved understanding of RBFs design

    - High dosing frequency resulted in an improved BOD_5 removal;
    - HLRs between 0.12 and 0.20 m^3/m^2/day did not impact RBFs performance significantly; and
    - Recycle ratios between 2:1 and 4:1 did no impact RBFs performance significantly.

4.5 Phase II – Post Factorial Analysis Investigation

4.5.1 Experimental Design

Based on the results of Phase I, a second series of bench-scale RBFs trials were conducted (Trial #17-21) to investigate in further detail the potential impact of dosing frequency on RBF performance. The five dosing frequencies studied in Phase II were: 48 (Trial #17), 96 (Trial #18), 144 (Trial #19), 192 (Trial #20), and 240 (Trial #21) times per day. The other RBF operating parameters were maintained constant with a HLR of 0.12 $m^3/m^2/d$ and recycle ratio of 4:1. For the Phase II trials, clean and dry crushed glass was used as the filter media at a depth of 15 cm in the bench-scale RBFs.

4.5.2 Results and Discussion

Development of RBFs

The average BOD_5 concentration in the septic tank effluent was 168 ± 21 mg/ L. Figure 4.6 indicates that a pseudo-steady-state condition in terms of BOD_5 removal in the bench-scale RBFs was achieved after three weeks of operation. It was observed that BOD_5 concentrations in the septic tank effluent varied from 132 mg/ L (minimum) to 220 mg/ L (maximum) in steady state condition (Figure 4.6). However, the measured effluent BOD_5 concentration after the initial 21-day start-up period was within a smaller range from 3 mg/ L (minimum) to 16 mg /L (maximum). This data indicates that the bench-scale RBFs were able to achieve a very stable effluent BOD concentration in spite of being supplied a highly variable influent wastewater stream. The average measured effluent BOD_5 concentration of the bench-scale RBF was 9 ± 5 mg/ L during the steady-state period. The other RBFs in Phase II had a similar trend as the crushed glass filter with dosing frequency of 96 times per day, as shown in Appendix: B – Biofiltration Effluent – BOD_5. The following discussion is based on the performance of the RBF systems during the steady-state period. In particular, Appendix B shows the start data point (marked with *) employed for system performance evaluation and comparison.

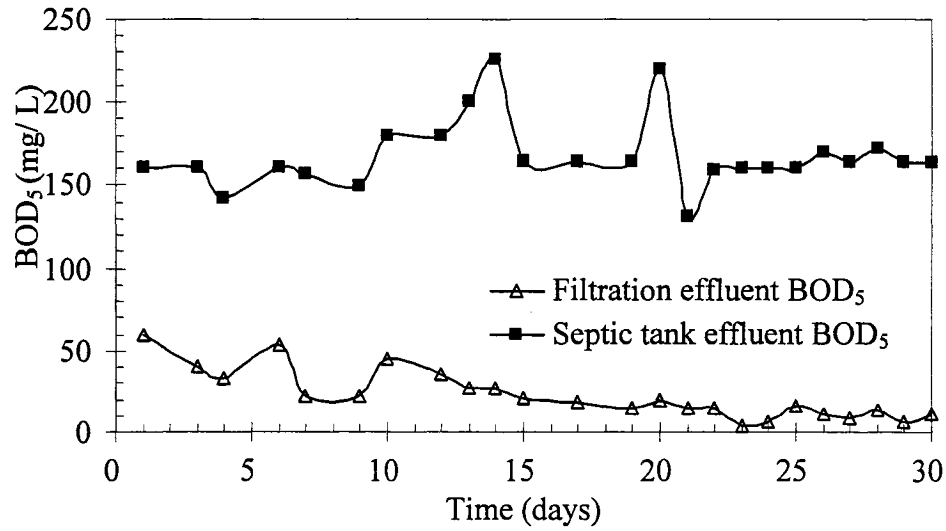


Figure 4.6 Development of the bench-scale RBF with the dosing frequency of 96 times per day

RBFs Performance

Currently, HAR is commonly applied to measure the depth of water introduced to the bed with each dose. Equation 4.1 shows that HAR is determined by recirculation ratio, HLR, and dosing frequency. However, the factorial analysis in Phase I demonstrated that only the dosing frequency had a significant impact on RBFs performance for BOD_5 removal at a confidence level of 95 %. It would be easier and more direct to design RBFs only based on dosing frequency rather than HAR. Therefore, Phase II discussed and compared RBFs performance only based on five different dosing frequencies.

BOD_5 Removal. Figure 4.7 shows that the effluent BOD_5 concentrations were 16 ± 11, 9 ± 5, 7 ± 6, 5 ± 1, and 4 ± 3 mg/ L for RBFs operated at dosing frequencies of 48, 96, 144, 192 and 240 times per day, respectively. It was observed that BOD_5 removal increased when dosing frequency increased from 48 to 192 times per day. However, when dosing frequency was 240 times per day, the effluent BOD_5 did not decrease as

compared to the effluent BOD_5 under dosing frequency of 192 times per day. In addition, the effluent BOD_5 from the RBF operated at a dosing frequency of 240 times per day was not as consistent as the effluent BOD from the RBF operated at a dosing frequency of 192 times per day. Therefore, a dosing frequency of 192 times per day was deduced to be the optimal dosing frequency based on these results of the bench-scale RBF trials.

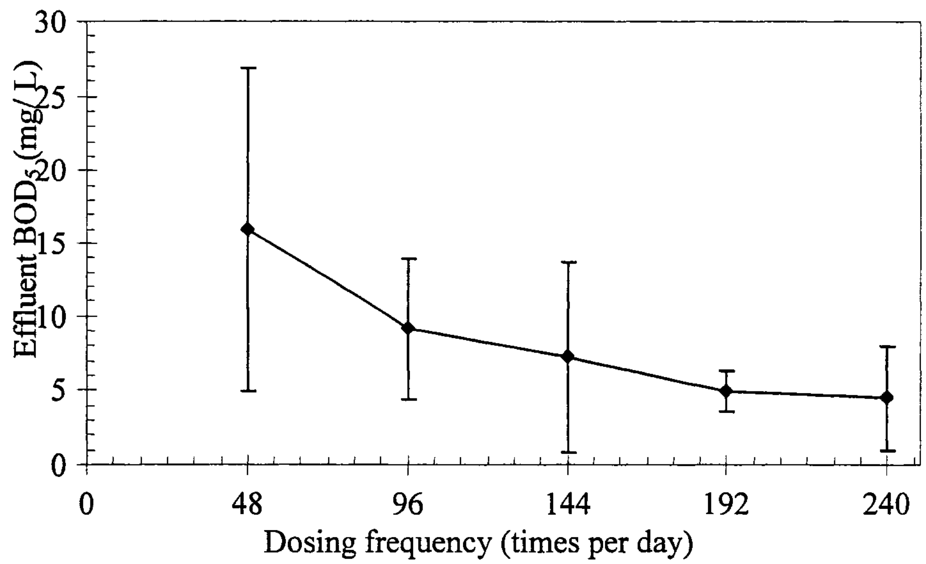


Figure 4.7 Impact of dosing frequency on BOD_5 removal (*Error bars indicate standard deviation from mean*).

Figure 4.8 presents a conceptual model of the biofiltration process (Loudon et al., 2001). This model suggests that the volume of liquid and the amount of nutrients and biodegradable substrates determines RBF system performance. If the volume per dose is too high, void space within the filter bed will be filled by water. Therefore, limited air can be diffused into the filter bed to support the aerobic biodegradation and biomass growth. Under these operating conditions it is less likely that a high BOD_5 removal can be achieved. If the volume per dose is too low, more void space will be kept open, allowing for more air (oxygen) to move into the filter bed. However, under these operating conditions the biomass/biofilm growth is also limited due to the lower volume of organic substrate supplied to the filter bed. Under these operating conditions, substrate removal, (i.e., BOD_5 removal) can be limited.

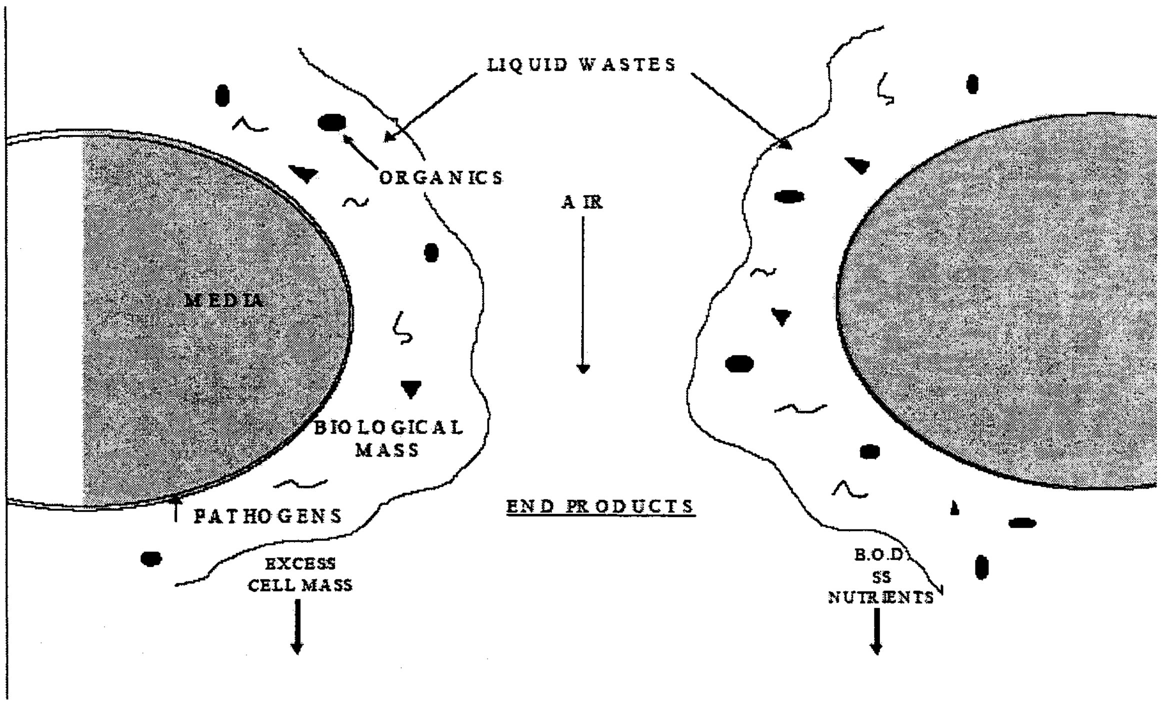


Figure 4.8 Biofiltration processes (Adapted from Loudon et al., 2001)

This conceptual model supports the experimental results of the present study. Specifically, the RBF operated with a dosing frequency of 240 times per day did not provide BOD_5 removal as stable as the RBF with the dosing frequency of 196 times per day as the RBF operated with a dosing frequency of 240 times per day had average effluent BOD5 with a higher standard deviation than the RBF operated with a dosing frequency of 196 times per day.

NH_4^+-N Removal. Figure 4.9 shows the RBF effluent NH_4^+-N concentrations for the five different dosing frequencies investigated in Phase II. The average effluent NH_4^+-N concentrations were 0.6 ± 0.1, 0.8 ± 0.2, 0.7 ± 0.2, 0.6 ± 0.3, and 0.5 ± 0.2 mg/ L for the bench-scale RBFs operated at a dosing frequency of 48, 96, 144, 192 and 240 times per day, respectively. In general, it was observed that the effluent NH_4^+-N concentration decreased with increasing dosing frequency. This trend is not consistent with the observation of BOD_5 removal presented in Figure 4.7. The improved NH_4^+-N reductions with increasing dosing frequency may be related to the low volume of wastewater per dose from high dosing frequency that allowed for the introduction of more air (oxygen) into the filter bed than operations at a lower dosing frequency. Therefore, improved

NH_4^+-N reductions were observed in the RBFs operated at a higher dosing frequency than at a lower dosing frequency.

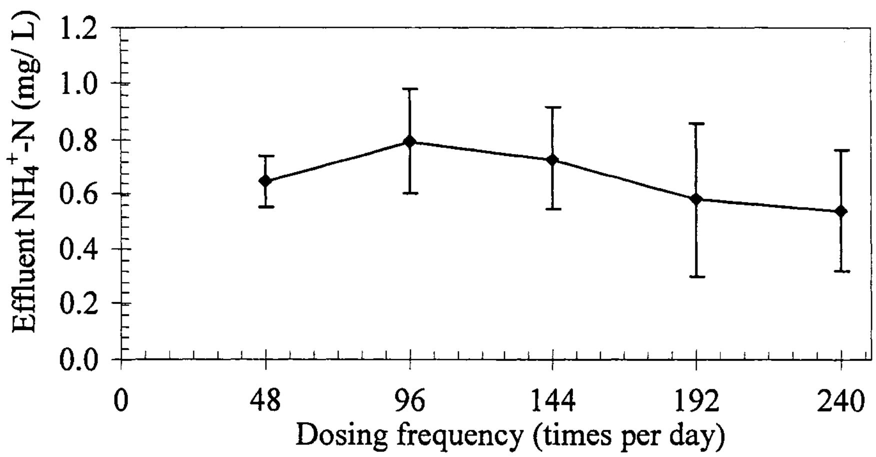


Figure 4.9 RBFs effluent NH_4^+-N under different dosing frequencies (*Error bars indicate standard deviation from mean*).

TN Removal. Figure 4.10 shows the effluent TN concentrations for the bench-scale RBFs operated at different dosing frequencies. The average effluent TN were 14.2 ± 2.7, 13.6 ± 1.0, 16.6 ± 3.7, 19.1 ± 4.7, and 18.3 ± 4.3 mg/ L for the RBFs operated at a dosing frequency of 48, 96, 144, 192 and 240 times per day. As presented in Figure 4.10 operation of the RBFs at a higher dosing frequency resulted in a higher effluent TN concentration than at a lower dosing frequency. Since RBF filter beds are designed as aerobic reactors, an apparent TN concentration reduction is not expected to occur within the filter beds. USEPA (2002) has reported that most TN removal occurs in the recirculation tank. Therefore, Phase II of this chapter did not discuss the possible reasons for different TN removal among these five dosing frequencies. TN removal in the recirculation tank will be deeply discussed in Chapter 5, 6, and 7.

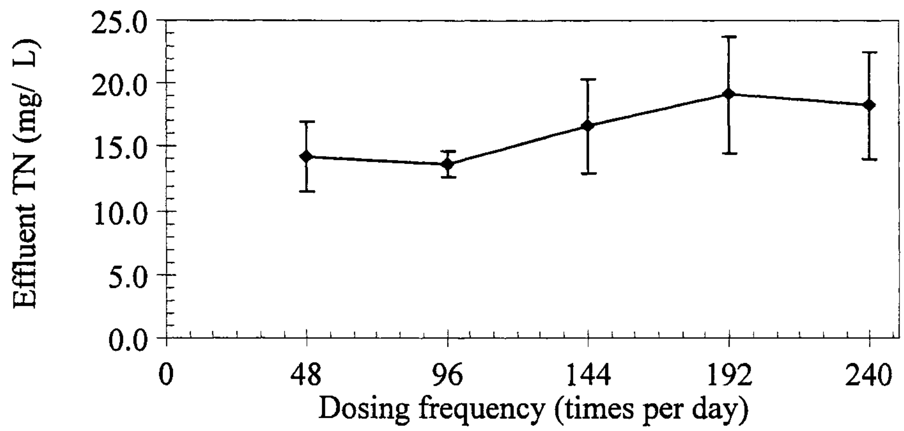


Figure 4.10 RBFs effluent TN over different dosing frequencies (error bars indicate standard deviation from mean)

Turbidity Removal. Figure 4.11 shows the effluent turbidity from the RBFs operated at different dosing frequencies. Turbidity is a measurement of the clarity of water. Although it is predominately used for potable water, it is also occasionally used to assess the performance wastewater treatment processes. Turbidity is an indirect measurement of the amount of suspended solids in the water. The average turbidity in the bench-scale RBFs effluents were 2.2 ± 0.5, 2.1± 1.1, 4.4 ± 2.1, 4.0 ± 2.1, and 1.9± 1.2 NTU for the dosing frequencies of 48, 96, 144, 192 and 240 times per day, respectively.

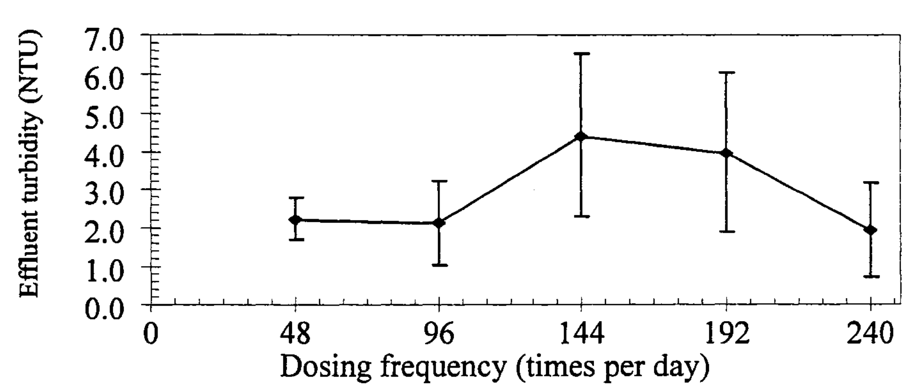


Figure 4.11 RBFs effluent turbidity over different dosing frequencies (error bars indicate standard deviation from mean)

4.5.3 Phase II Summary

In summary, the results of Phase II bench-scale studies suggest:

1. Operating RBFs at higher dosing frequencies (i.e., 192 times per day) can improve BOD_5 removal. However, there may be a maximum dosing frequency (i.e., 240 times per day) beyond which no further BOD_5 removal can be achieved.
2. Operating RBFs at higher dosing frequencies may result in improved NH_4^+-N removal.
3. Operating RBFs at higher dosing frequencies may result in lower TN removal as compared to operating at a lower dosing frequency.

4.6 Phase III – Filter Media Comparison

4.6.1 Experimental Design

Four bench-scale RBFs were conducted in Phase III to compare four filter media, namely silica sand (Trial #22), crushed glass (Trial #23), peat (Trial #24), and geotexitle (Trial #25). These four RBFs were operated under identical operating conditions, including HLR of 0.12 $m^3/m^2/d$, a recycle ratio of 4:1 and a dosing frequency of 96 times per day.

4.6.2 Development of RBFs

Figure 4.12 indicates that a pseudo-steady-state condition in terms of BOD_5 removal was achieved after five weeks operation. The average BOD_5 concentration of the septic tank effluent was 164 ± 12 mg/ L during the steady-state period. It was observed that BOD_5 concentration of the raw water varied from 150 mg/ L (minimum) to 180 mg/ L (maximum) under steady-state operating conditions (Figure 4.12). However, the effluent BOD_5 concentration after the initial 31-day start-up period was within a smaller range from 2 mg/ L (minimum) to 14 mg /L (maximum). This data indicates that the bench-scale RBFs were able to achieve a very stable effluent BOD_5 concentration in spite of variable BOD_5 loadings in the influent wastewater. The average effluent BOD_5 concentration of the bench-scale sand RBF (Trial #22) was 7 ± 4 mg/ L during the steady-state period. The following discussion on the Phase III trials is based on the recorded water quality during the steady-state period, as reported in Appendix: C.

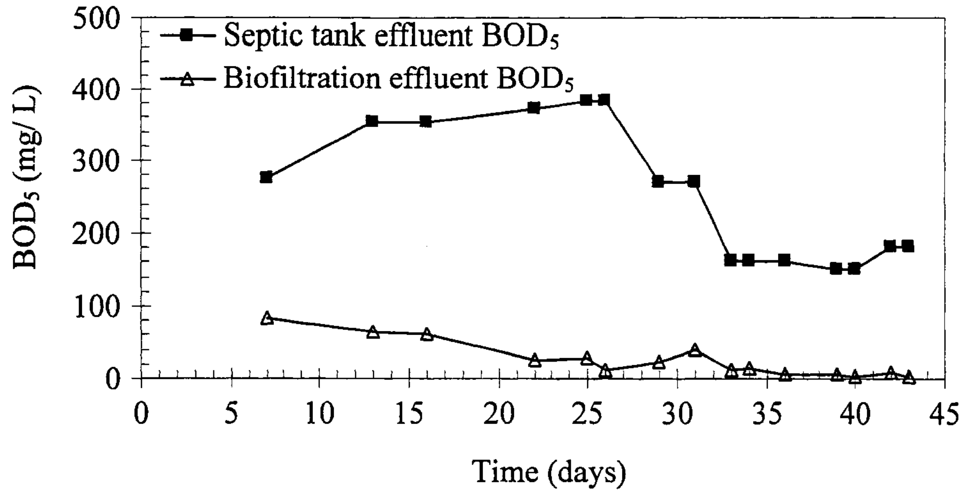


Figure 4.12 Development of the sand RBF

4.6.3 RBFs Performance

BOD_5 Removal. The average BOD_5 removals of RBFs were 96 ± 2%, 92 ±2 %, 93 ± 4%, and 97 ± 3% for the silica sand, crushed glass, peat and geotextile bench-scale RBFs, respectively. Phase III found that the peat RBF could provide BOD_5 removal greater than 90 % with an effluent BOD_5 of 13 ± 5 mg/ L. One possible reason for the high BOD_5 removal observed in the peat filter was that peat has a high capacity to bind water (Ebeling et al. 2002), which results in a longer retention time in the filter. Overall, the removal of BOD_5 in the bench-scale RBFs of Phase III was very close to the data reported in literature (Table 4.3). Even though, all biofilters provided BOD_5 concentration in the biofilter effluent that was less than 20 mg/ L for all media.

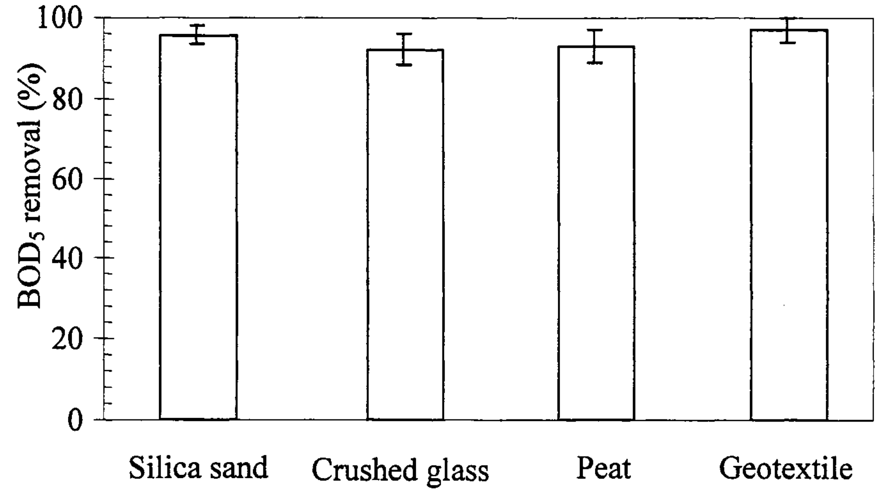


Figure 4.13 Effluent BOD_5 concentrations from bench-scale (*Error bars indicate standard deviation from mean*).

Turbidity Removal. Figure 4.14 shows the effluent turbidity in the biofiltration effluents for all media. The average effluent turbidity in the effluent samples of the bench-scale RBFs were 1.9 ± 0.4 NTU, 1.5 ± 0.1 NTU, 3.1 ± 0.9 NUT, and 1.1 ± 0.2 NTU for the silica sand, crushed glass, peat and geotextile media, respectively. As shown in Figure 4.14, the peat filter resulted in the highest effluent turbidity as compared to the other three RBFs. During the trials, it was observed that the peat filter effluent was brown in color. This visual observation indicates that natural decay of the peat possibly occurred during experimental period as inspection of the other three RBFs effluents showed a treated stream with good clarity.

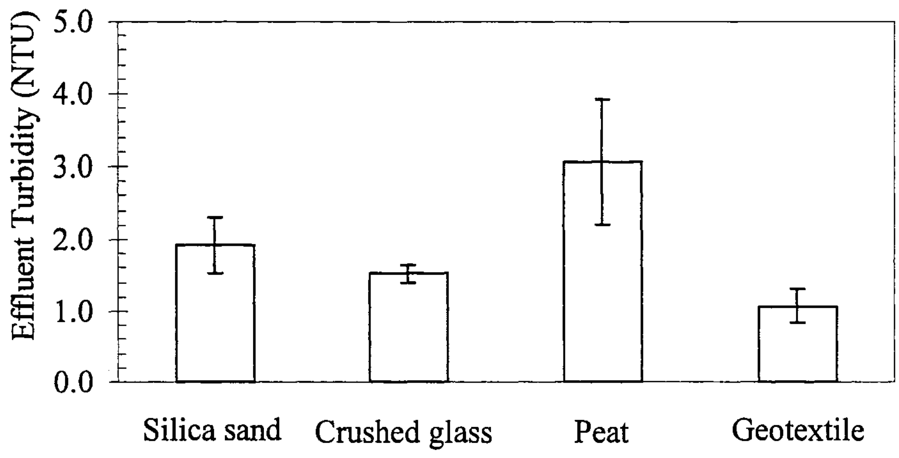


Figure 4.14 RBFs effluent turbidity (*Error bars indicate standard deviation from mean*).

NH_4^+-N Removal. Figure 4.15 shows the effluent NH_4^+-N concentrations measured in the bench-scale RBF effluent samples. The average effluent NH_4^+-N concentrations were 0.9 ± 0.8 mg/ L, 0.7 ± 0.5 mg/ L, 5.5 ± 2.3 mg/ L, and 0.7 ± 0.5 mg/ L for silica sand, crushed glass, peat and the geotextile filters, respectively. As presented in Figure 4.15, the peat filter produced an effluent NH_4^+-N concentration much higher than the other three RBFs. Brooks et al. (1984) reported on two peat biofilters (single-pass) for residential wastewater treatment in Maine with effluent NH_4^+-N concentrations of 10.4 and 17.7 mg/ L. Leveren et al. (2001) reported on two peat biofilters operated in Minnesota with effluent NH_4^+-N concentrations of 25.0 and 19.0 mg/ L for single pass and multiple pass biofilters, respectively. The effluent NH_4^+-N concentrations reported in the Brooks et al. (1984) and Leveren et al., (2001) studies are much higher than the NH_4^+-N concentrations found in the present study using a bench-scale peat RBF. Based on the literature reports as well as the present study result, it can be concluded that peat biofilters (both single- and multi-pass) are likely to provide reduced NH_4^+-N removal as compared to RBFs designed with other filter media.

Although nitrate concentrations (NO_3^--N) in the RBFs effluent samples were not recorded, this study assumes that nitrification is the principle mechanism for NH_4^+-N removal. It has been suggested in literature that NH_4^+-N removal within biofilter beds is primarily achieved through the nitrification process (USEPA, 2002). Decrease of pH can be applied as an indicator of nitrification process (Rittmann and MaCarty, 2001). Therefore, this chapter investigated the pH values in the biofiltration effluents for all media, as shown in Figure 4.15. The average pH in the bench-scale RBF effluents were 7.9 ± 0.1, 7.4 ± 0.2, 4.0 ± 0.5, and 6.8 ± 0.9 for the silica sand, crushed glass, peat and geotextile RBFs, respectively. As shown in Figure 4.15, the peat filter in the Phase III trials produced an effluent pH much lower than the other RBFs. The optimal pH value for nitrification is recommended to be between 7.2 and 9.0 (Metcalf & Eddy, 2003). Based on the data presented in Figure 4.15, the low NH_4^+-N removal by the peat filter is consistent with the low effluent pH of the peat filter. This observation is consistent with the results of other studies. In the Brooks et al. (1984) study, the effluent pH ranged from 5.3 to 6.5, with effluent NH_4^+-N concentrations of 10.4 and 17.7 mg/ L.

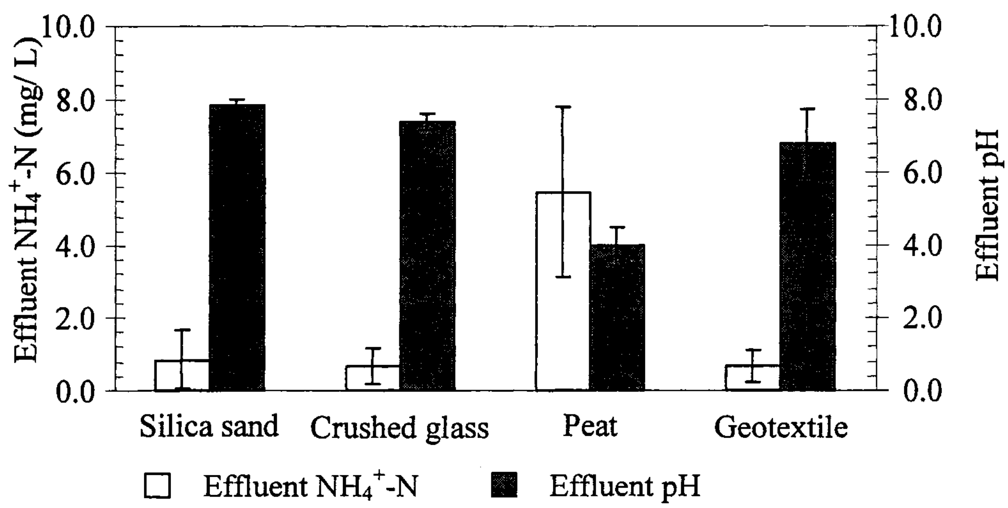


Figure 4.15 RBFs effluent NH_4^+-N and pH (*Error bars indicate standard deviation from mean*).

TN Removal. TN removal varied considerably between the four bench-scale RBFs. Overall TN removals for sand and crushed glass (52.5 % and 63.3 %, respectively) were lower than geotextile and peat filters (84.4 % and 69.4 %, respectively). The results of sand and crushed glass are consistent with previous research (Table 4.3). The TN removal results of the peat and geotextile RBFs in the present study were higher than the values in Table 4.3. Denitrification requires an anaerobic environment and an adequate food source for denitrifying bacteria to convert nitrate to nitrogen gas. A number of studies have explored the concept of modified recirculating biofilters to improve nitrogen removal from wastewater. In addition, USEPA (2002) reported that most of the denitrification occurs in the recirculating tank.

Table 4.3 Comparison of RSFs performance from previous studies.

Reference	BOD_5 (mg/L)		TSS (mg/L)		Fecal coliform (#/100 mL)		TN (mg/L)	
	Inf	Eff	Inf	Eff	Inf	Eff	Inf	Eff
Louden et al., 1985[a]	150	6	42	6	3.40E+03	1.40E+01	55	26
Piluk and Peters, 1994[b]	235	5	75	8	1.80E+06	9.20E+03	57	20
Ronayne, et al., 1982[c]	217	3	146	4	2.60E+05	8.50E+03	57.5	31.5
Roy and Dube, 1994[d]	101	6	77	3	4.80E+05	1.3E+04	37.7	20.1
Owen and Bobb, 1994[e]	80	8	36	6	n/a		n/a	

[a] Single-family home filters. Sand media: d_{10}=0.3mm; UC=4.0. Average loadings = 0.036 m^3/m^2 per day. Recirculation ratio=3:1. Doses per day = 96-144.
[b] Single-family home filters. Sand media: d_{10}=1mm; UC=<2.5. Design hydraulic loadings = 1.42 m^3/m^2 per day. Recirculation ratio=3:1 to 4:1. Doses per day = 24.
[c] Single-family home filters. Sand media: d_{10}=1.2mm; UC =2.0. Maximum hydraulic loading =0.12 m^3/m^2 per day. Recirculation ratio = 3:1 to 4:1. Doses per day = 48.
[d] Single-family home filters. Gravel Media: d_{10}=4.0mm; UC =<2.5. Design hydraulic loading = 0.94 m^3/m^2 per day. Recirculation ratio=5:1. Doses per day = 48. Winter operation.
[e] Small community. Sand media: d_{10}= 1.5 mm; UC = 4.5. Design hydraulic loading = 1.10 m^3/m^2 per day. Recirculation ratio = 1:1 to 4:1. Winter operation.

TP Removal. Crushed glass, geotextile, and peat did not produce an effluent that would meet the regulation of CWQG for a total phosphorus discharge less than 1.0 mg/ L. The average effluent TP concentrations were 5.4 ± 2.8 mg/ L, 2.2 ± 1.7 mg/ L, and 3.8 ± 3.3 mg/ L for crushed glass, geotextile, and peat RBFs, respectively. However, geotextile medium was close to this requirement with an effluent TP concentration of 1.1 mg/ L. These results may due to the fact that geotextile media has a larger porosity (i.e., surface area in the filter) which has been suggested to improve TP adsorption in biofiltration (USEPA, 2002).

Fecal Coliform Removal. All of the bench-scale RBFs showed good removal of fecal coliform bacteria with statistically similar average log reductions of 2.5. These results are consistent with the results of other studies that found 2.0 to 3.0 log reductions in fecal coliforms in RSFs (USEPA, 2002).

4.6.4 Phase III Summary

In summary, the results of Phase III suggest that:

1. Geotextile could be considered as an innovative medium for RBFs;
2. Peat is not an effective medium for RBFs due to the poor removal of NH_4^+-N and the low effluent pH; and
3. RBFs cannot provide efficient TP removal.

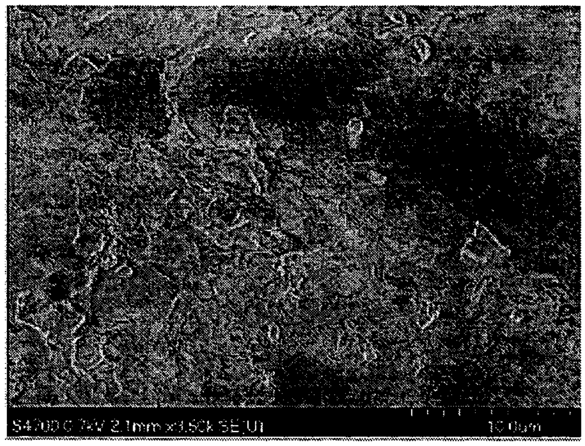
4.7 Phase IV – Biomass and Filter Depth Study

4.7.1 Organic Loading Rates

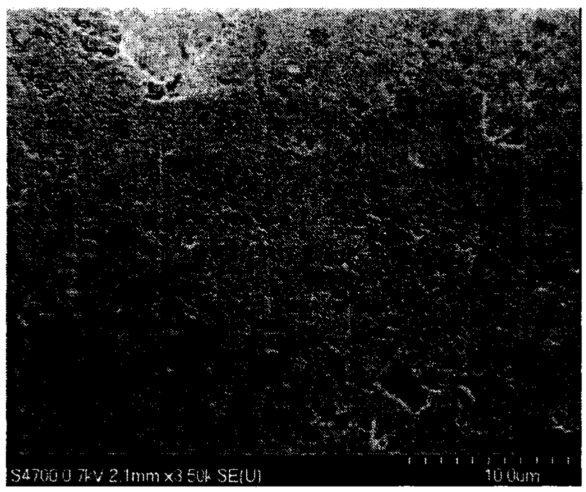
Organic loadings to RBFs is a critical design parameter because the amount of organic material added to the filter with each dose must be such that the microorganisms in the biofilm can process the mass of organic matter added without accruing additional mass between doses (Crites and Tchobanoglous, 1998). Table 4.1 shows that high HLRs result in high organic loading rates under the same raw wastewater BOD_5 concentrations. In this phase of the research project, the organic loading rate was 0.034 $kg/m^2/d$ in some trials under a HLR of 0.20 $m^3/m^2/d$ (Table 4.1), which is much higher than the loading value of 0.024 $kg/m^2/d$ recommended by the USEPA (2002). However, visual observations during the short-term (i.e., 2 months) period of the Phase III trials did not show any clogging occurrences in the bench-scale RBFs. In order to gain more comprehensive understanding of the impact of organic loading rates on RBF treatment performance, more longer-term studies should be conducted.

4.7.2 Biomass Profile

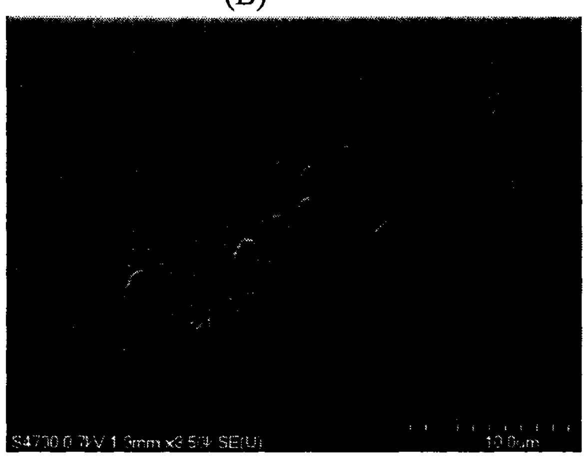
Since the biofilm formed on the filter media is the main microbial component to utilize organic matter present in the wastewater, scanning electronic microscope (SEM) images were taken to further investigate the structure of biofilm at the different depths of the biofilters. Figure 4.16 shows the structure of biofilm around particles in the RCGFs (Trial 9) at different filter depths, including the top, middle, and bottom of the filter bed. The images reveal that the biofilm on media particles at the top of the filter bed had a much higher density than the biofilm on media particles in the middle and bottom sections of the filter bed. In particular, the biofilm development at the bottom of the filter consisted of sporadic individual bacteria instead of a biofilm as shown in Figure 4.16.C. This observation is consistent with conclusions of the USEPA (2002) that most of the biochemical treatment occurs within approximately 15 cm of the filter surface. In addition, Furman et al. (1995) and Calaway et al. (1952) reported that purification processes occurred primarily within the top 20 to 30 cm of the media depth. Anderson et al. (1985) reported that additional media depth imparted consistency, assuring a more uniform effluent quality. Therefore, Phase IV involved an evaluation of a pilot-scale crushed glass RBF with a filter bed depth of 30 cm.



(A)



(B)



(C)

Figure 4. 16 SEM images of biofilm around particles (Trial 9) at: A) top of the filter; B) middle of the filter; C) bottom of the filter

4.7.3 Filter Depth

The 30-cm pilot-scale crushed glass RBF (Trial 26) was conducted in parallel with the dosing frequency studies of Phase II. The detailed pilot-scale RBF setup has been previously described in Chapter 3. In Phase IV, the HLR, dosing frequency and recycle ratio for the pilot-scale RBF were 0.12 $m^3/m^2/d$, 96 times per day and 4:1, respectively, which are consistent with Trial #18 in Phase II. Therefore, Phase IV investigated filter depth by comparing the 30 cm pilot-scale crushed glass RBF (Trial #26) to the bench-scale crushed glass RBF with dosing frequency of 96 times described in Phase II (or Trial #18). The comparison was conducted based on the RBF effluent water quality parameters, including BOD_5, NH_4^+-N, TN, and turbidity.

BOD_5 Removal. Figure 4.17 shows the BOD_5 concentrations measured in samples taken from the septic tank, the 15-cm bench-scale RBF and the 30-cm pilot-scale RBF effluents. It was observed that the 30-cm pilot-scale RBF produced a lower effluent BOD_5 concentration than the 15-cm bench-scale RBF during the first week of operation. Figure 4.17 demonstrates that both the 15-cm bench- and the 30-cm pilot-scale RBFs achieved steady-state operations after three weeks. Appendix D shows the start data point employed for system performance evaluation and comparison, as marked with an asterisk. The average effluent BOD_5 concentration during the steady-state period were 11 ± 6 and 10 ± 4 mg/ L for the 15-cm bench- and 30-cm pilot-scale RBFs, respectively. Paired t-tests showed that there was no significant difference between the 15-cm and 30-cm RBF systems in terms of effluent BOD_5 concentrations during the steady-state period (α=0.05), as shown in Table 4.5.

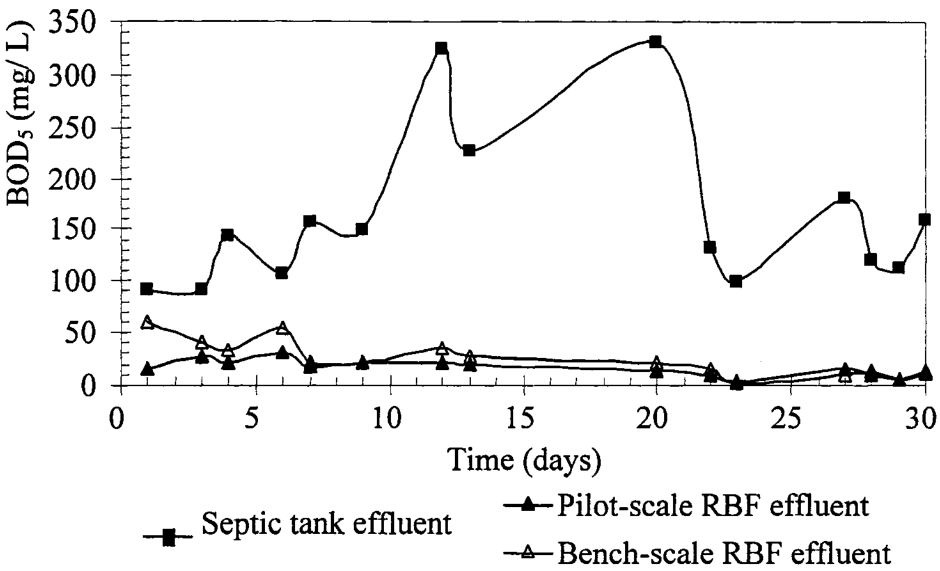


Figure 4.17 Effluent BOD_5 concentrations of septic tank, bench- and pilot-scale RBF effluents

Turbidity removal. Figure 4.18 shows turbidity concentrations in the effluents of the 15-cm bench- and 30-cm pilot-scale RBFs. The average turbidity in the RBF effluents were 1.9 ± 0.4 and 2.7 ± 0.2 NTU for the bench- and pilot-scale RBFs, respectively during the steady state period. During the initial 20-day start up period, the 30-cm pilot-scale RBF produced an effluent with lower turbidity than the 15-cm bench-scale RBF. However, the 15-cm bench-scale RBF produced an effluent with lower turbidity than the 30-cm pilot-scale RBF during the steady-state operational period. However, paired t-tests showed that there was no significant difference between the 15-cm bench- and 30-cm pilot-scale RBFs in terms of the recorded effluent turbidity during the steady-state period, as shown in Table 4.5.

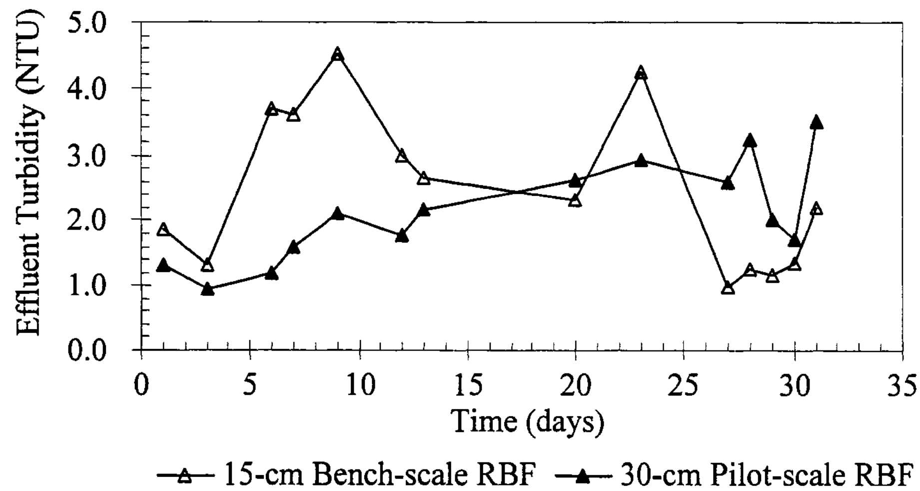


Figure 4.18 Effluent turbidity of bench- and pilot-scale RBFs.

NH_4^+-N Removal. Figure 4.19 shows the NH_4^+-N concentration in the effluent of the 15-cm bench- and the 30-cm pilot-scale RBFs. The average effluent NH_4^+-N concentrations were 0.8 ± 0.2 and 1.0 ± 0.4 mg/ L for the bench- and pilot-scale RBFs, respectively during the steady-state period. Paired t-tests showed that there was no significant difference between these two RBFs in terms of effluent NH_4^+-N (α=0.05), as shown in Table 4.5.

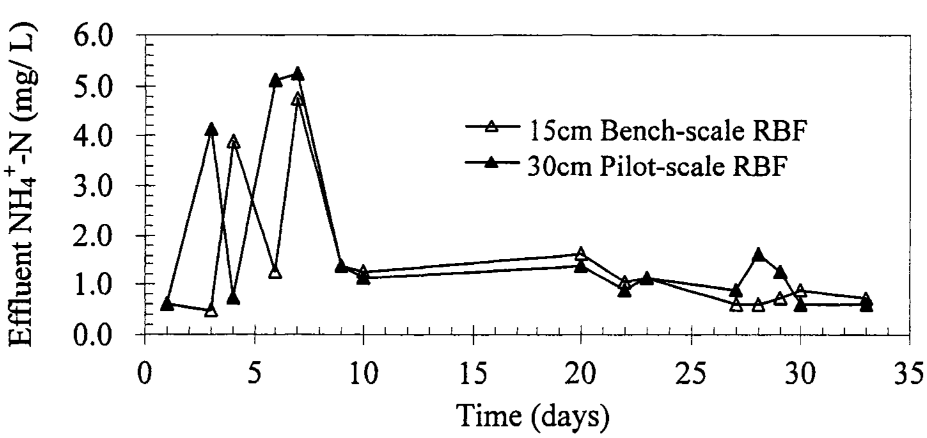


Figure 4.19 Effluent NH_4^+-N of the bench- and pilot-scale RBFs

TN Removal. Figure 4.20 shows the TN concentrations in the effluents of the 15-cm bench- and the 30-cm pilot-scale RBFs. The overall average effluent TN concentrations were 13.6 ± 1.0 and 15.0 ± 2.0 mg/ L for the bench- and pilot-scale RBFs, respectively during the steady-state period. The average TN removals were 71 % and 66 % for the bench- and pilot-scale RBFs, respectively. However, paired t-tests showed that there was no significant difference between these two RBFs in terms of effluent TN during the steady-state period (α=0.05), as shown in Table 4.5.

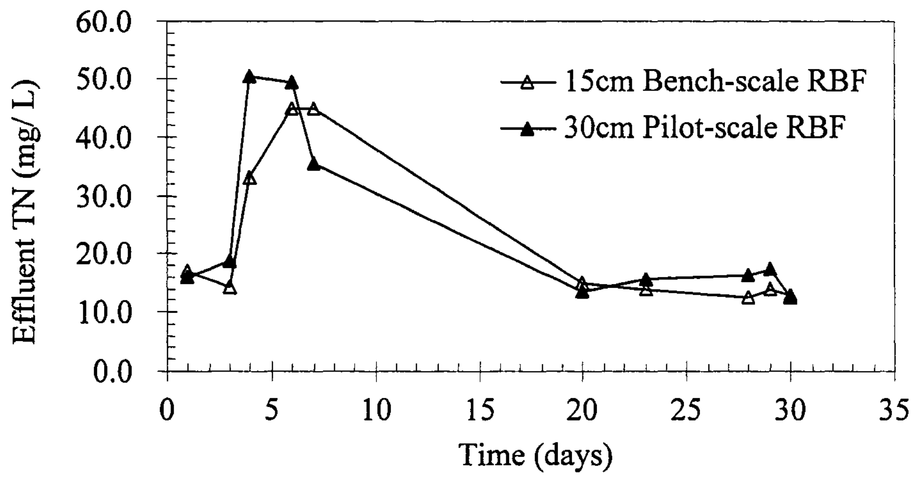


Figure 4.20 Effluent TN of the bench- and pilot-scale RBFs

Table 4.4 Summary of statistical analysis (paired t-test) between the 15cm bench- and the 30cm pilot-scale RBFs performance (α=0.05)

Parameter	Significant difference	No significant difference
BOD_5		√
Turbidity		√
NH_4^+-N		√
TN		√

The detailed paired t-test calculation is shown in Appendix I.

4.7.4 Phase IV Summary

In summary, the results of Phase IV suggest:

1 RBFs can be operated under higher than USEPA (2002) recommended organic loadings based on a short-term bench-scale study;

2 Biofilm development on the media was not evenly distributed throughout the filter bed depth. Lower filter media particles surface coverage by biofilm was observed through SEM test in Phase IV; and

3 There were no significant differences between a 15-cm bench- and a 30-cm pilot-scale RBFs for biofiltration effluent water quality parameters (α=0.05), including BOD_5, turbidity, NH_4^+-N and TN. Therefore, a 15-cm bench-scale RBF can be applied to simulate the performance of a pilot-scale, or even a field-scale RBF in a short-term study. This bench-scale RBF model will provide considerable advantages, since bench-scale RBFs can significantly shorten process development time. Furthermore, the results of Phase IV indicated that short-term bench-scale RBFs can be applied as a RBF design tool for the variation of wastewater characteristic.

4.8 Conclusions

This chapter examined the impacts of various impact factors that may influent RBFs performance. These parameters included filter media, HLR, recycle ratio, and dosing frequency. The examination of these impact factors was conducted through four phases. Phase I found that dosing frequency was a significant factor for BOD_5 removal in RBFs based on a 2^4 factorial analysis. In addition, Phase I confirmed that crushed glass could be an effective alternative option to sand for a RBF medium, since the 2^4 factorial analysis found that there was no significant difference between crushed glass and sand filters for BOD_5 removal.

Based on the conclusion of Phase I, Phase II conducted another five bench-scale RBFs with five different dosing frequencies, including 48, 96, 144, 192, and 240 times per day. The results showed that a high dosing frequency could result in an improved removal efficacy for BOD_5 than a low dosing frequency. However, BOD_5 removal wouldn't increase under a very high dosing frequency, such as 240 times per day.

Phase III compared four types of filter media, including silica sand, crushed glass, peat, and geotextile. The results repeatedly found that crushed glass performed similarly as sand. This finding could help to identify a possible secondary market for recycled glass. Crushed glass is a readily available media in developed countries where pollution prevention initiatives have promoted the recycling of certain manufacturing products. However, this chapter found that the peat filter was not able to achieve the requirement of CWQG for the poor NH_4^+-N removal and the low effluent pH. In addition, peat produced a yellow-color effluent, which would be undesirable from an aesthetic perspective. Geotextile was also examined as a possible filter medium in this chapter, although the geotextile had much larger porosity and surface area than sand, crushed glass, and peat.

Phase IV discussed the organic loading rates, biofilm profiles around the filter media over different depth, and the optimal filter depth for bench-scale RBFs which could be applied as a short-term model to simulate and predict large-scale RBFs performance. The results showed that RBFs could be dosed with higher than USEPA (2002) recommended organic loading rates based on a short-term study. SEM tests found that biofilm did not distribute evenly over the entire filter bed. No noticeable biofilm was observed around the crushed glass at the bottom of filter beds. Therefore, Phase IV conducted a 30cm pilot-scale RBF as compared to a 15cm bench-scale RBF based on biofiltration water quality, including BOD_5, turbidity, NH_4^+-N, and TN. Paired t-tests found that there were no significant differences between these two RBFs in terms of these four water quality parameters in the effluents. Therefore, Phase IV suggested that a short-term 15cm bench-scale RBF could be applied to simulate and predict the performance of a pilot-scale, even a field-scale RBF.

5. IMPACT OF OXYGEN SUPPLY ON RBF PERFORMANCE

5.1 Abstract

The main objectives of this chapter were (1) to compare three different ventilation locations for RBFs including at the top, the bottom sidewall, and top and bottom sidewall of the filter beds; (2) to investigate the impact of ventilation location on the performance of RBFs for the removal of BOD_5, turbidity, NH_4^+-N, TN, and fecal coliform counts; and (3) to construct a conceptual model of air flow for RBFs ventilated at different locations. The results of this study showed that the removal of BOD_5 could be significantly improved by venting the RBF from both the top and bottom sidewall of the filter bed. However, it was found that there was no significant difference in the removal of turbidity, NH_4^+-N or fecal coliforms with these three different ventilation locations. In addition, this chapter discussed the function of the recirculation tank as a combined reactor for both denitrification and aerobic BOD_5 removal. The results showed that ventilation at the surface of the RBF was more optimal for denitrification in the recirculation tank than when the system was ventilated from the bottom sidewall of the filter bed. Based on these results, two conceptual air flow models for RBFs ventilated from the top surface and the bottom sidewall of the filter beds are provided in this chapter.

5.2 Introduction

5.2.1 Overview of Air Flow in Porous Media

In unsaturated, fine-grained soils, oxygen transport is generally controlled by molecular diffusion (Collin 1987; Collin and Rasmuson 1988). When the degree of saturation is below about 85–90%, this diffusion essentially occurs in the air phase (Aubertin and Mbonimpa 2001). Above these values, however, the air phase becomes discontinuous and diffusion flux occurs through the water-filled voids (Corey 1957). In the latter case, the amount of oxygen that can diffuse through the porous media is limited by the maximum concentration of oxygen in water ($Cw \approx 9.2$ mg/L), which is about 30 times less than the equilibrium concentration of oxygen in air ($Ca \approx 276.7$ mg/L) at 20 °C (Mbonimpa et al., 2003).

Fick's laws are commonly used to evaluate diffusive air transport in both the aqueous phase (Freeze and Cherry 1979; Shackelford 1991) and the gaseous phase (Troeh et al. 1982; Reible and Shair 1982; Reardon and Moddle 1985; Jin and Jury 1996; Aachib 2002). As the free diffusion coefficient of oxygen is about four orders of magnitude larger in air than in water, diffusive transport of oxygen in the water-filled pores is much slower than that in the air-filled voids. A layer that remains close to full saturation impedes the passage of oxygen as a barrier layer (Collin 1987; Nicholson et al. 1989).

5.2.2 Background of Ventilation for RBFs

Rotating Biological Filters (RBFs) are aerobic, fixed-film bioreactors. Air within the pores of the media contains oxygen, which is transferred into the thin film of water on the particles by diffusion. Organisms attached to surfaces within the film can then use the oxygen so that the dominant biological activity is that of aerobic organisms digesting contaminates in the wastewater as the effluent moves slowly through the system (Loudon and Lindsay, 2006). Most RBFs are constructed aboveground with an open filter surface. This design provides ample fresh air venting due to reaeration of the filter media from the filter surface (USEPA, 2002). Aeration of the filter bed can be achieved by either active or passive aeration mechanisms. To achieve active aeration, a fan is installed at the under drainage system or at the top of the filter bed to maintain oxygen levels within the filter bed (McCarty et al. 2001). Passive aeration is widely applied in RBF design. Air ventilation at the top surface and the bottom sidewall of the filter beds was explored in this thesis.

Packed bed filters are unsaturated units in which the air diffuses in and through the voids created between the media. Reaeration of the filter medium primarily occurs from the filter surface with air diffusion into the filter bed from the atmosphere (Converse 2001). The surface of a recirculating sand filter must be free of any soil cover so that there is free air movement through the unit. During RBF operation, the lower 20% of the medium's depth maintains high moisture content with medium layers at the bottom of the filter near or at saturation (USEPA, 2002). Under these conditions, a barrier to air flow from the underdrain system is realized. At these deeper depths in the media, organism populations are reduced, oxygen may be less available and reaction rates are lower (Loudon et al., 2003). Therefore, the gravel surrounding the distribution piping must be vented to the surface to provide a fresh air flow (USEPA, 2002).

Air transfer within the media of a RBF can occur both due to diffusion from the top of the filter surface down through the media and due to convection from the underdrain system upward through the media of the filter (Loudon and Lindsay, 2006). Air is also drawn in as the wastewater moves through the media (State of Wisconsin, 1999). Improving oxygen exchange within the filter can be realized by increasing the dose frequency and/or including a ventilation system in the filter with vents extended to the atmosphere (Washington State Department of Health, 1999).

The main objectives of this chapter were:

- to compare three different methods to maintain aerobic condition in the filter bed, including ventilation at the top surface, bottom sidewall, and top & bottom of the filter beds;
- to investigate the impact of ventilation location on the performance of RBFs for the removals of organic compounds (as quantified by BOD_5 and TSS concentrations), nutrients (as quantified by NH_4^+-N and TN concentrations) and pathogenic microorganisms (as quantified by fecal coliform counts); and
- to construct a conceptual air flow model under different ventilation designs.

5.3 Materials and Methods

5.3.1 Description of Laboratory RBFs

To evaluate the impact of oxygen supply location on the performance of RBFs, this study was conducted using three pilot-scale RBFs. The overall design of RBFs is shown in Figure 5.1.A. Figure 5.1.B presents a detailed sketch of the filter column used in the pilot-scale RBF systems.

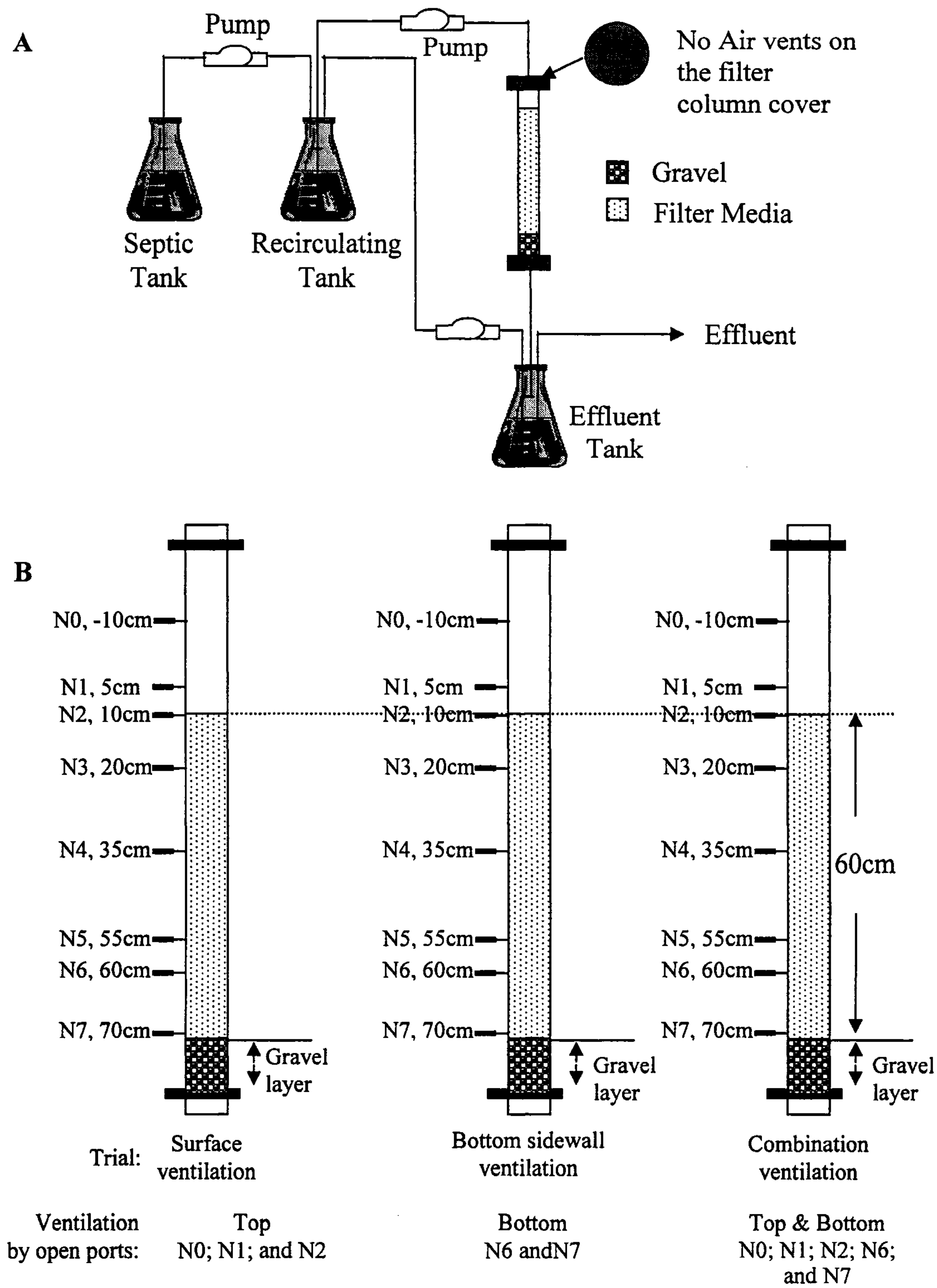


Figure 5.1 Pilot-Scale RBF Experimental Design (A) complete RBFs system and (B) detailed filter bed ventilation design.

The pilot-scale RBF filter columns were much more flexible in terms of filter bed depth adjustment than the column of bench-scale RBFs, since seven sampling ports existed over the entire depth of the column as shown in Figure 5.1.B. For example, the filter column could be loaded with media up to location N1, which provided 70 cm of filter depth. In addition, the design of pilot-scale filter columns for this research allowed for investigations into the impact of ventilation location on system performance. In particular, the supply of air-oxygen at the surface, the bottom sidewall and both top and bottom of the filter beds was controlled by having ports of variable depth open to the atmosphere.

In this chapter, all three pilot-scale RBFs were operated under a hydraulic loading rate (HLR) of 0.12 $m^3/m^2/d$, recycle ratio of 4:1, and dosing frequency of 96 times per day. The RBFs in this study were filled with dry, clean crushed glass which had been shown to be an effective filter medium in Chapter 4. Each RBF was filled with crushed glass between ports N2 and N7 to provide a 60-cm operating depth, with a 5-cm layer of gravel at the bottom. Bench-scale RBF trial #1 was operated under surface ventilation by having ports N0, N1 and N2 open to the atmosphere. Bench-scale RBF trial #2 was operated under bottom sidewall ventilation by having ports N6 and N7 open to the atmosphere. Bench-scale RBF trial #3 was operated under surface and bottom ventilation by having ports N0, N1, N2, N6 and N7 open to the atmosphere.

5.3.2 Sampling and Water Quality Parameters Measurements

Samples were collected from the septic tank, recirculation tank, and effluent tank. The samples were analyzed for BOD_5, TN, NH_4^+-N, NO_3-N, pH, fecal coliform, and turbidity using methods previously described in Chapter 3. The samples were collected 2 to 3 times a week at the first three weeks and 3 to 5 times per week for the duration of the experiment.

5.4 Results and Discussion

5.4.1 RBF Acclimation Period

The results of the experiments showed that a pseudo-steady-state operating condition was achieved after approximately three weeks for all of the RBFs. Figure 5.1 shows the effluent BOD_5 concentration over time from the surface ventilation RBF with ventilation provided at the surface of the filter bed. The results of the other two ventilation designs showed similar development in effluent BOD_5 as Figure 5.2 (Appendix: E – Biofiltration Effluent – BOD_5).

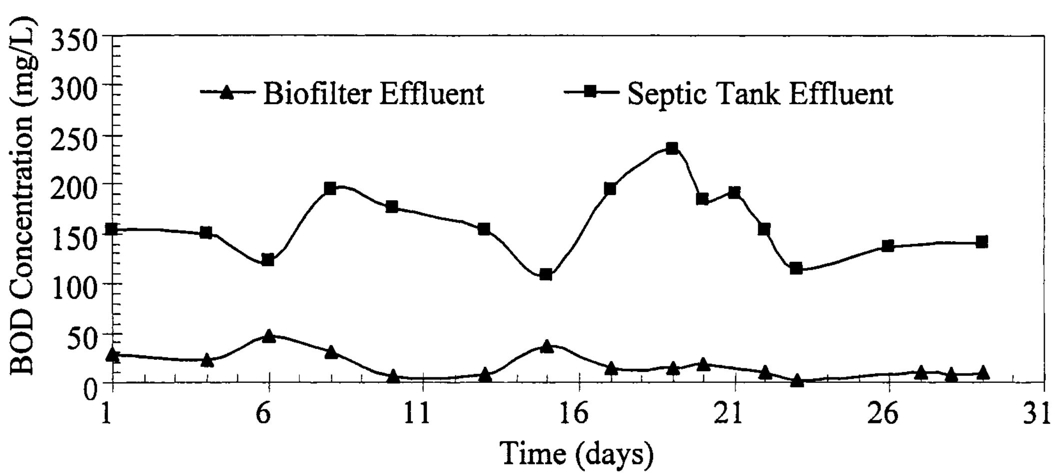


Figure 5.2 Effluent BOD_5 from the surface ventilation RBF

The raw water BOD_5 from the septic tank varied from 156 mg/ L (minimum) on Day 23 to 236 mg/ L (maximum) on Day 19 (Figure 5.2). Regardless of the highly variable influent BOD_5 concentration, the effluent BOD_5 concentration from the surface ventilation RBF was within a small range from 3 mg/ L (minimum) on Day 24 to 19 mg/ L (maximum) on Day 21. These results demonstrate the treatment robustness of RBFs as they were able to achieve a very stable effluent quality throughout the course of the experiments. The discussion in next sections of this chapter is based on the system performance after the pseudo-steady-state was achieved. In particular, Appendix E shows the start data point employed for systems performance evaluation and comparison with a character of an asterisk.

5.4.2 Contaminant Removal in Pilot-Scale RBFs

Treatment in the recirculating crushed glass filter was achieved through a combination of physical, chemical and biological transformations. Suspended solids were removed principally by mechanical straining due to chance contact with the crushed glass particles and sedimentation. The colonization of bacteria on the surface of the crushed glass grains further enhances the removal of suspended solids by auto filtration caused by the growth of bacteria. Specific chemical constituents were removed by chemical and physical sorption on the grains of the crushed glass. There are two biological processes that govern the removal of organic and chemical constituents from the RBF feed-stream. The removal of soluble BOD_5 was achieved by the oxidation of organic material by microorganisms present in the crushed glass bed and occurs under aerobic conditions. The biological removal of ammonium can be achieved through nitrification, where ammonium is sequentially oxidized to nitrite (NO_2^-) and nitrate (NO_3^-) by two primary groups of autotrophic nitrifying bacteria (e.g., *Nitrosomonas* sp. and *Nitrobacter* sp.).

***BOD_5* Removal**

The average measured BOD_5 concentrations after biofiltration were 10 ± 6, 20 ± 7, and 3 ± 1 mg/ L for surface ventilation, bottom sidewall ventilation, and combined ventilation RBFs, respectively. ANOVA results showed that there was a significant difference among the three RBFs in terms of BOD_5 removal (α=0.05), as shown in Appendix: I – ANOVA – Ventilation – BOD_5. As presented in Figure 5.3, the surface ventilation RBF resulted in a decrease in average BOD_5 from 163 mg/ L to less than 15 mg/ L. However, the bottom sidewall ventilation RBF with ventilation provided at the bottom sidewall of the filter bed could only produce an average effluent BOD_5 of less than 30 mg/ L. The combined ventilation RBF resulted in the largest removal of organics with an average effluent BOD_5 of 3 mg/ L after the pseudo-steady-state period achieved. These results indicate that ventilation location in the filter bed is an important factor in BOD_5 removal.

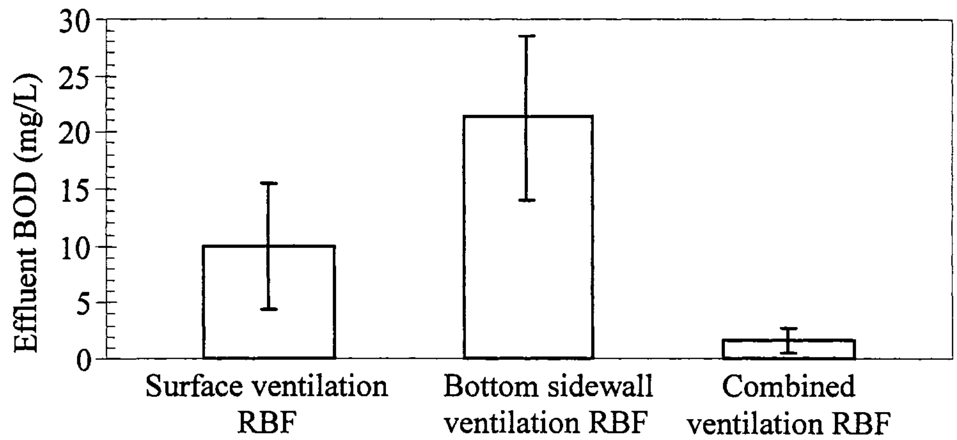


Figure 5.3 Average effluent BOD_5 from pilot-scale RBFs (Error bars indicate standard deviation from mean)

Turbidity Removal

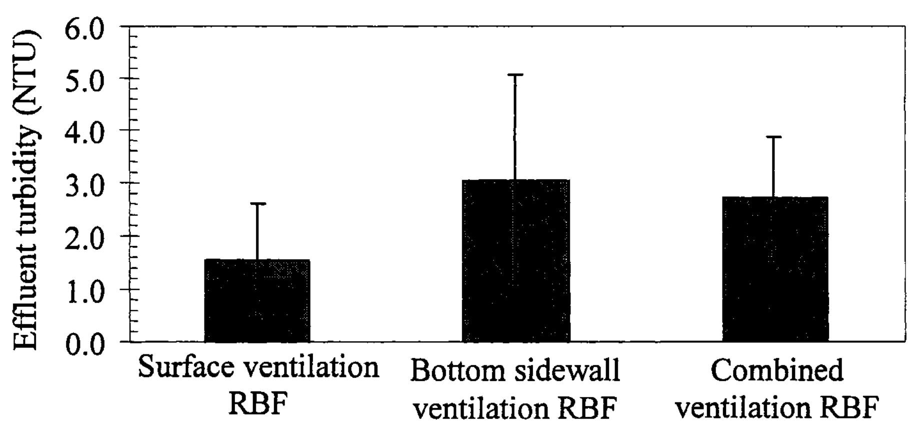


Figure 5.4 Average effluent turbidity from pilot-scale RBFs (Error bars indicate standard deviation from mean)

The average measured turbidity concentration after biofiltration were 1.5 ± 1.1, 3.0 ± 2.0, and 2.7 ± 1.2 mg/ L for surface ventilation, bottom sidewall ventilation, and combined ventilation RBFs, respectively, as shown in Figure 5.4. ANOVA results showed that there was no significant difference among the three RBFs in terms of turbidity removal at the confidence level of 95%, as shown in Appendix: J – ANOVA – Ventilation - Turbidity. Figure 5.4 shows that bottom sidewall ventilation RBF could not produce a stable effluent turbidity as compared to surface ventilation and combined ventilation RBFs.

Nitrification

Nitrification is a two-step biological process in which ammonium (NH_4-N) is sequentially oxidized to nitrite (NO_2-N) and nitrate (NO_3-N) by two primary groups of autotrophic nitrifying bacteria (e.g., *Nitrosomonas* sp. and *Nitrobacter* sp., respectively). Nitrification is an autotrophic, aerobic process in which energy for bacterial growth is derived by the oxidation of inorganic carbon (e.g., carbon dioxide). Nitrifier cell yield per unit of substrate metabolized is smaller than the cell yield for heterotrophs that use organic carbon for the synthesis of new cells. The following equation is an overall balanced reaction for the complete oxidation of ammonium (NH_4^+-N) to nitrate (NO_3^--N) by nitrifiers having a retention time of 15 days (Rittmann and McCarty, 2001)

$$NH_4^+ + 1.8\ O_2 + 0.13\ CO_2 \rightarrow 0.026\ C_5H_7O_2N + 0.97\ NO_3^- + 0.92\ H_2O + 1.97H^+$$

Nitrification occurs most efficiently when the pH of the wastewater is in the range of 7.2 to 9.0 (Metcalf and Eddy, 2003). In this study, the average pH values of the filter influents ranged between 7.5 and 7.7 for the three RBF systems. The average filter effluent pH values ranged between 7.7 and 7.9 for the three RBF systems. Although these values were on the lower end of the optimum pH range for nitrification, the operating conditions within the pilot-scale RBF systems presented a suitable environment for nitrification to occur.

The average measured NH_4^+-N concentrations after biofiltration were 0.5 ± 0.8, 0.4 ± 0.7, and 1.2 ± 0.7 mg/ L for surface ventilation, bottom sidewall ventilation, and combined ventilation RBFs, respectively. Figure 5.5 shows that the average effluent NH_4^+-N concentration of the combined ventilation RBF was slightly higher than measurements taken from the other two RBFs effluent streams. However, ANOVA results showed that there was no significant difference among the three RBFs in terms of

NH_4^+-N removal at the confidence level of 95%, as shown in Appendix: J – ANOVA – Ventilation – NH_4^+-N.

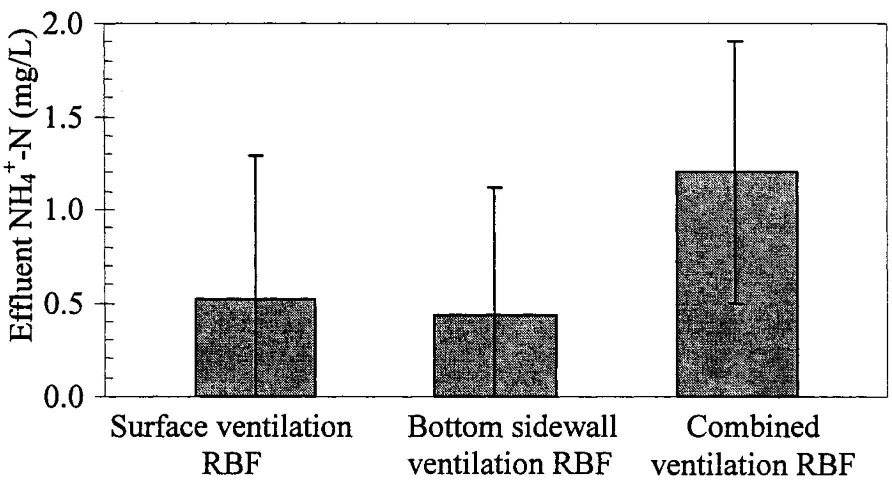


Figure 5.5 Average effluent NH_4^+-N from pilot-scale RBFs (Error bars indicate standard deviation from mean)

Pathogenic Microorganisms Removals

The average measured fecal coliform counts after biofiltration were 108 ± 193, 184 ± 245, and 400 ± 520 colonies/ 100mL for surface ventilation, bottom sidewall ventilation, and combined ventilation RBFs, respectively. Figure 5.6 shows that the combined ventilation RBF produced effluent fecal coliform slightly higher than the other two RBFs. However, ANOVA results showed that there was no significant difference among the three RBF systems in terms of fecal coliform removal at the confidence level of 95%, as shown in Appendix: J – ANOVA – Ventilation – Fecal coliforms.

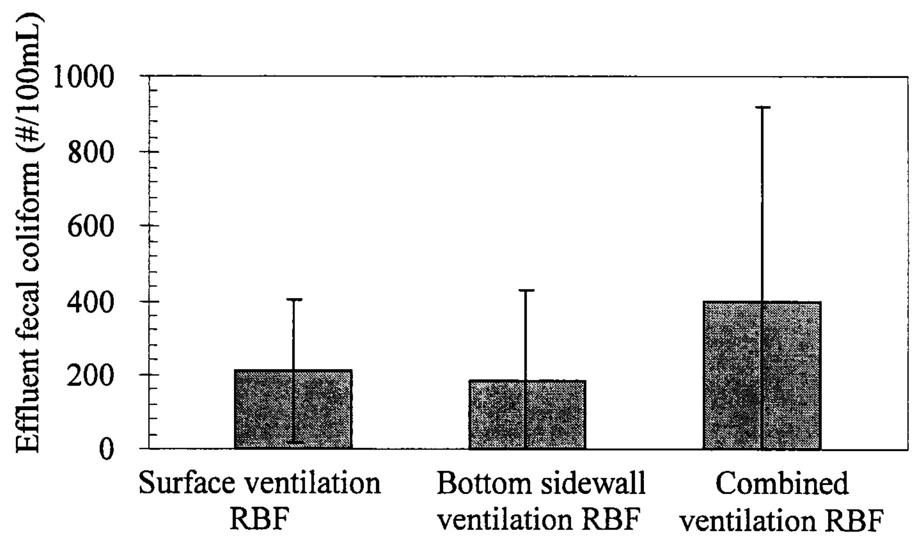


Figure 5. 6 Average effluent fecal coliform from pilot-scale RBFs (Error bars indicate standard deviation from mean)

5.4.3 Chemical Reactions within the Recirculation Tank

Based on the results of this study, it is proposed that two chemical processes occurred in the recirculation tank during the pilot-scale RBF trials — aerobic removal of BOD_5 and denitrification. The prevailing conditions (such as oxygen concentration in the recirculation tank) determine the favoured process. This study did not measure the water quality in the recirculation tank when the RBF was ventilated from both the top and bottom sidewall of the filter bed. Therefore, the following discussion on proposed removal mechanisms within the recirculation tank is based only on results from the trials involving the surface ventilation and the bottom sidewall ventilation RBFs.

Denitrification

USEPA (2002) has suggested that most of the denitrification process occurs in the recirculation tank, with average TN removal between 40 to 60%. Optimal environmental conditions for denitrification include maintaining a moderate temperature between 20 and 35°C and operating within a pH range of 7 to 8 (Metcalf & Eddy, 2003). The same reference suggests a pH range of 6.5 to 7.5, with 7.0 as the optimum pH using 1975 EPA study data. During the pilot-scale RBF trials, the temperature of effluent samples collected from the recirculation tank averaged 20°C. Samples collected from the recirculation tank during the same experimental period showed an average pH of 7.5 and 7.7 for the surface ventilation and the bottom sidewall ventilation RBFs, respectively. The following equation is an overall balanced reaction that represents the general denitrification process (Mosley 2001):

$$0.15\ C_{10}H_{19}O_3N + NO_3^- + H^+ \rightarrow 0.105\ C_5H_7O_2N + 0.45\ N_2 + 0.15\ NH_4^+ + 0.15\ HCO_3^- + 0.81\ CO_2 + 1.13\ H_2O$$

In this equation $C_{10}H_{19}O_3N$ represents the composition of wastewater, the electron donor or the carbon energy source. As proposed by Barth et al., (1968) approximately 4 g ultimate carbonaceous BOD (BOD_L) is required for each gram of NO_3^-N that is reduced. The average total nitrogen (TN) concentration measured in septic tank effluent samples taken during the pilot-scale RBF trials was 38.1 ± 8.9 mg/ L. However, each time that the wastewater recirculated through the filter it increased the concentration of dissolved oxygen in the recirculation tank. Mosley (2001) reported that dissolved oxygen concentrations in the recirculation tank increased from approximately 1 mg/ L to about 5 mg/ L due to the high dissolved oxygen in the filter effluent. In that study, natural air circulation in the recirculating sand filter was provided by a vented under drain system without the expense of energy demanding aeration systems. Therefore, the complete TN removal in the recirculation tank was not theoretically achievable.

Evidence of the occurrence of denitrification in the recirculation tank can be found in the data collected during the pilot-scale RBF trials. Specifically, the trends of total nitrogen (TN), nitrate (NO_3^-N) and BOD_5, measurements taken from samples of the recirculation tank effluent support this theory. A mass balance on the recirculation tank was conducted using septic tank and filter effluent quality concentrations and flow rates to determine the projected concentration of these constituents in the recirculation tank. This balance is presented schematically in Figure 5.7.

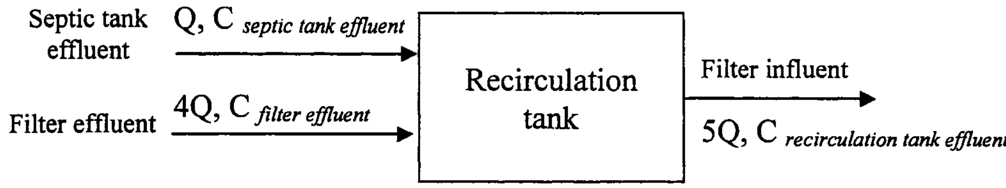


Figure 5. 7 Schematic mass balances in the recirculation tank

Based on this balance, projected concentrations of BOD_5, total nitrogen and nitrate were determined by the equation 5.1.

$$C_{recirculating\ \tan k} = \frac{C_{filter\ \ effluent} \times 4 + C_{septic\ \ \tan k\ \ effluent}}{5} \quad [5.1]$$

Where,

$C_{recirculation\ tank}$ is the water quality parameter concentration in recirculation tank
$C_{filter\ effluent}$ is the water quality parameter concentration in filter effluents
$C_{septic\ tank\ effluent}$ is the water quality parameter concentration in septic tank effluent

For example, based on a filter effluent BOD_5 measurement of 21 mg/L and a septic tank effluent BOD_5 measurement of 153 mg/ L with a recirculation ratio of 4:1, one would project using Equation 5.1 a recirculation tank effluent BOD_5 concentration of $(21\times4 + 153)/5$ or 47 mg/ L. This projected concentration would be expected in the recirculation tank effluent stream if no biological action occurred in this process tank. Therefore, this chapter investigated the function of the recirculation tank by comparing between projected and actual water quality in the recirculation tank.

As presented in Figure 5.8, there was a difference in projected and the actual total nitrogen (TN) concentrations in the recirculation tank during both the surface ventilation and the bottom sidewall ventilation trials, indicating that total nitrogen removal was occurring the recirculation tank. In particular, the degree of TN removal in the recirculation tank was more pronounced for surface ventilation RBF than that for bottom sidewall ventilation RBF, as shown in Figure 5.8. This observed was possibly caused by the different available oxygen in the recirculation tanks, which will be discussed in detailed in Section 5.5.4.

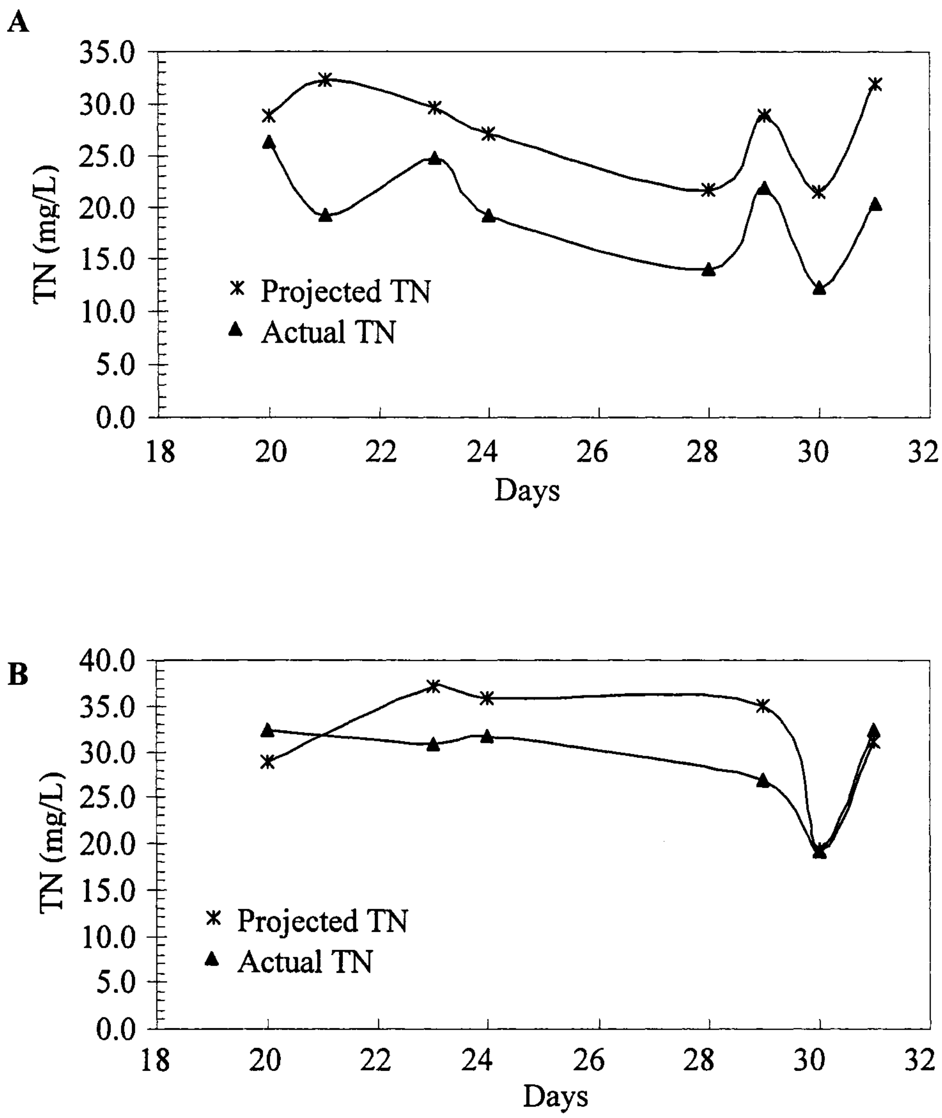
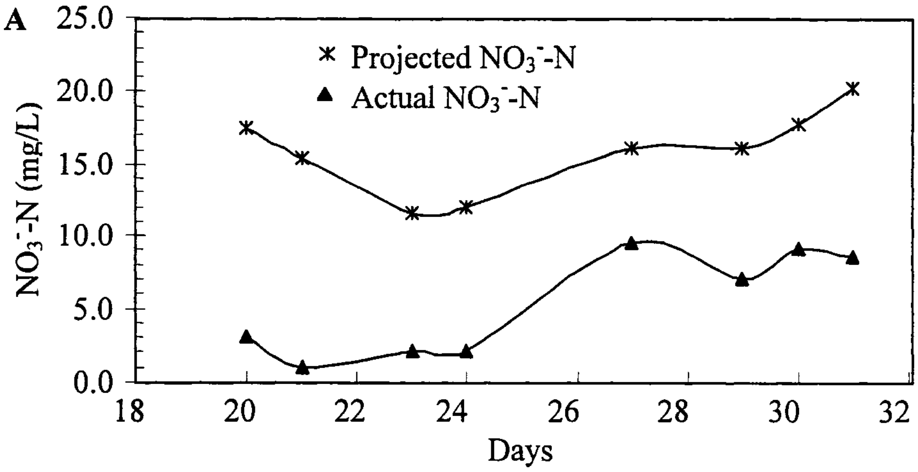


Figure 5.8 Estimation of TN removal between projected and the measured values in the recirculation tank: A) surface ventilation RBF and B) bottom sidewall ventilation RBF

In the surface ventilation RBF, the average projected TN into and the actual measured TN leaving the recirculation tank were 27.8 mg/ L and 19.8 mg/ L, respectively. Based on these data, TN was reduced by 29 % in the recirculation tank. In the bottom sidewall ventilation RBF, the average projected TN into and the actual measured TN leaving the recirculation tank were 31.2 mg/ L and 28.9 mg/ L, respectively. Based on these data, an 8 % reduction of TN in the recirculation tank of the bottom sidewall ventilation RBF was achieved. These data suggest that the recirculation tank of the RBF ventilated from the bottom sidewall of the filter bed could not provide an environment for TN removal as optimal as the recirculation tank of the RBF ventilated from the surface of the filter bed. This is discussed in more detail later in this chapter under the section dealing with the conceptual air flow model.

Figure 5.9 shows the concentration of nitrate (NO_3^-N) measured in the recirculation tank during the surface ventilation RBF and the bottom sidewall ventilation RBF trials. The overall decreasing concentrations of nitrate through the trials provide additional support that denitrification was occurring in the recirculation tank. Figure 5.9.A shows the projected and actual measure NO_3^--N in the recirculation tank of the surface ventilation RBF. The average projected and actual measure concentrations were 15.8 and 5.3 mg/ L, respectively. Based on these data, nitrate was reduced by 66 % in this RBF system. Figure 5.9.B shows the projected and actual measured NO_3^--N in the recirculation tank of the bottom sidewall ventilation RBF. The average projected and actual measured concentrations were 18.6 and 12.8 mg/ L, respectively. Based on these data, a 31 % reduction of nitrate in the bottom sidewall ventilation RBF was achieved. These results were consistent with the TN removal data in the surface ventilation RBF and the bottom sidewall ventilation RBF systems found previously. In the recirculation tank of the bottom sidewall ventilation RBF, a lower nitrate removal was observed as compared removals achieved in the recirculation tank of the surface ventilation RBF. The reduction in total nitrogen and nitrate in both recirculation tanks suggest that denitrification was occurring in the recirculation tanks and nitrate was biologically reduced to nitrogen gas.



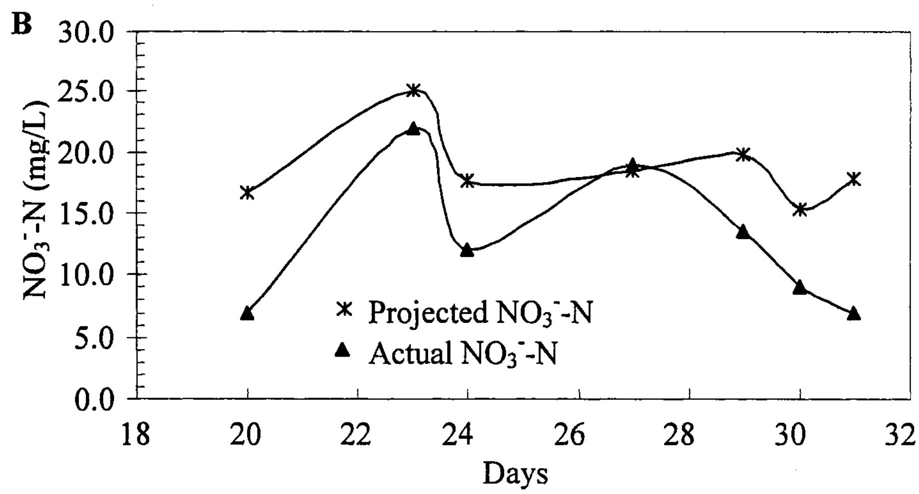


Figure 5.9 Estimation of NO_3^--N removal between projected and the measured values in the recirculation tank: A) surface ventilation RBF and B) bottom sidewall ventilation RBF.

Aerobic Removal of BOD_5

As described in the previous sections, the aerobic removal of BOD_5 possibly occurred in the recirculation tank due to the increase in oxygen concentration of filter effluents. Typical reported dissolved oxygen (DO) concentration from various published literature varies from 2 to 6 mg/L. Loudon et al. (2003) reported that the average filter effluent DO was 3-5 mg/ L. However, that report did not specify which type of aeration (i.e., active or passive) was applied. Loudon and Lindsay (2006) reported that the effluent DO from recirculating sand filters (RSFs) is typically within the range of 3 to 6 mg/ L by providing air from top of the filter bed through convection and diffusion as well as by providing air from the drain system due to convective upward movement. The average biofiltration effluent DO from Orenco® System is approximately 2.6 mg/ L with a passive aeration design with venting at the surface of the filter bed). This addition of oxygen into the recirculation tank needs to be minimized as much as possible so that nitrate is used as the electron acceptor by denitrifiers rather than the preferential use of oxygen as an electron donor by heterotrophic bacteria. The following figure shows the projected and actual quantities of BOD_5 in the recirculation tank and indicates that BOD_5 removal occurred in the recirculation tanks of both the surface ventilation RBF and the bottom sidewall ventilation RBF.

Figure 5.10.A shows the projected and actual measured BOD_5 in the recirculation tank of the surface ventilation RBF system. The average projected and actual measured BOD_5 were 50 and 38 mg/ L, respectively. Based on this data, approximately 24 % of the BOD_5 was removed in the recirculation tank of the surface ventilation RBF system. Figure 5.10.B shows the projected and actual measured BOD_5 in the recirculation tank of the bottom sidewall ventilation system. The average projected and actual measured BOD_5 were 48 mg/ L and 23 mg/ L, respectively. Based on this data, approximately 52 % of the BOD_5 was removed in the recirculation tank of the bottom sidewall ventilation RBF system. The results show that more BOD_5 was removed in the recirculation tank of the bottom sidewall ventilation RBF system than the surface ventilation RBF system.

As presented in Table 5.1, total nitrogen, nitrate and BOD_5 removals in the recirculation tanks of the surface ventilation RBF and the bottom sidewall ventilation RBF were found to be different.

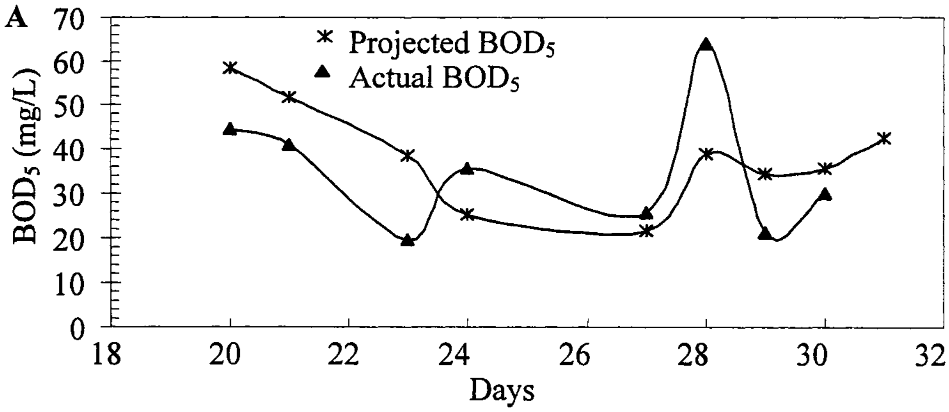


Figure 5.10 Estimation of BOD5 removal between projected and the measured values in the recirculation tank: A) surface ventilation RBF and B) bottom sidewall ventilation RBF

In particular, higher total nitrogen and nitrate removals and lower BOD_5 removals were observed in the surface ventilation RBF in comparison to the bottom sidewall ventilation RBF system. Denitrification is a complex biological process that occurs in the absence of or under limited dissolved oxygen concentrations (Metcalf & Eddy, 2003). In the presence of dissolved oxygen the biological removal of BOD_5 is achieved with heterotrophic bacteria using oxygen as the electron acceptor (Rittmann & McCarty, 2001). Both denitrification and BOD_5 removal are complex biological processes that are dependent on a number of environmental factors (e.g., pH, temperature, alkalinity, dissolved oxygen). However, the data in Table 5.1 hypothetically suggest that the recirculation tank of the surface ventilation RBF was operating with limited dissolved oxygen concentrations as compared to the recirculation tank of the bottom sidewall ventilation RBF, resulting in a higher TN and NO_3^--N removal and lower BOD_5 removal in the surface ventilation RBF system as compared to the bottom sidewall ventilation system. Based on these results and preliminary hypothesis, a conceptual model was developed to gain a better understanding of the movement of air within the RBF system and how RBF ventilation design could potentially impact organic and inorganic removal rates within these systems.

Table 5.1 Summary of removal rates in the recirculation tanks of the surface ventilation RBF and the bottom sidewall ventilation RBF.

Parameter	Recirculation Tank (surface ventilation RBF)	Recirculation Tank (bottom sidewall ventilation RBF)
TN	29	8
NO_3^--N	66	31
BOD_5	24	52

5.4.4 Conceptual Model of Air Flow through Pilot-Scale RBF Systems

The analysis of section 5.4.3 found that TN and NO_3^--N removals in the recirculation tank of the surface ventilation RBF system which was ventilated from the surface of the filter bed were higher than the recirculation tank of the bottom sidewall ventilation RBF system which was ventilated from the bottom sidewall of the filter bed. As well, the analysis found that the BOD_5 removal in the recirculation tank of the surface ventilation RBF system was lower than the recirculation tank of the bottom sidewall ventilation RBF system. These collective results of the RBF trials indicate that the recirculation tank of the surface ventilation RBF system was more optimal for the denitrification process than the recirculation tank of the bottom sidewall ventilation RBF system. Since the temperature and pH value in the recirculation tanks of both RBF systems were similar through the experimental trials, it is proposed that potential differences in oxygen concentrations in the recirculation tanks resulted in different removals of total nitrogen, nitrate and BOD_5 between the two RBF systems.

One of the remediation solutions for failed RBFs can be attempted by actively introducing air into the filter bed. Usually this would be the installation of an air manifold under the media layer. If needed, a fractional horsepower regenerative blower or compressor is connected to the manifold. Air is diffused into the media layer in an attempt to change the anaerobic environment back to an aerobic environment (Loudon and Lindsay, 2006). This solution is consistent with the concept provided by USEPA (2002) that the medium at the bottom of RBFs is near or at saturation, which is a barrier to air flow and venting from the underdrain system. As the free diffusion coefficient of oxygen is about four orders of magnitude larger in air than in water, diffusive transport in the water-filled pores is much slower than that in the air-filled voids (Mbonimpa, 2003). Therefore, oxygen is unlikely able to move upward through the filter bed when the RBF is ventilated from the bottom sidewall of the filter bed. Under this ventilation design, the bottom filter bed is near or at saturation conditions thereby restricting the diffusion of air through the water-filled pores. In this study, the average filter influent and effluent BOD_5 concentrations for the bottom sidewall ventilation RBF system were 23 and 20 mg/ L, respectively. These data demonstrate that limited aerobic BOD_5 removal occurred in the filter bed when the RBF was ventilated from the bottom sidewall of the filter bed.

Air is also drawn through the filter bed concurrently with the wastewater stream as it moves downward through the media (State of Wisconsin, 1999). This concept can be applied to the results of this study. In the bottom sidewall ventilation RBF system when ventilation was provided at the bottom sidewall of the filter bed, it is proposed that air was drawn into the bottom layers of the filter bed and directed with the flow of the effluent stream into the recirculation tank.

These airflow models can be applied to the total nitrogen, nitrate and BOD_5 reductions observed in the surface ventilation RBF and the bottom sidewall ventilation RBF systems. Specifically, BOD_5 removal in the recirculation tank of the bottom sidewall ventilation RBF was found to be 52 %, or more than double the BOD_5 reduction found in the recirculation tank of the surface ventilation RBF. These results indicate that higher DO concentrations were realized in the recirculation tank of the bottom sidewall ventilation RBF due to the concurrent flow of air with wastewater flow.

Based on data analysis from the present study and literature reports, the following conceptual air flow model for the pilot-scale RBF systems investigated in this project is presented. Due to the significant difference of the air diffusion coefficient in air and water (Mbonimpa, 2003), the conceptual air flow model is discussed at three distinct operating periods: (i) during the dosing period, (ii) after the completion of the dosing period, and (iii) between subsequent dosing periods .

<u>Surface Ventilation RBF</u>:

(i) During the dosing period (Figure 5.11.A). The recirculation tank effluent was dosed into the filter bed, and air diffusion through the filter bed was low due to the volume of wastewater filling the void spaces and the creation of a more saturated zone within the full depth of the filter bed. During this period, the oxygen already present inside the filter bed is utilized for the biodegradation of organic matter.

(ii) After the dosing operation (Figure 5.11.B). The recirculation tank effluent dosing pump was turned off and the wastewater volume begins to drain slowly downward through the filter bed. Air diffusion from the surface of filter bed commenced due to the increased void space within the upper layers of the filter bed due to the creation of a more unsaturated zone. During this period, the wastewater drained out of the filter bed, and the oxygen inside of the filter bed moved concurrently with the wastewater effluent flow into the recirculation tank. Therefore, the D.O in the recirculation tank was high, which could facilitate some aerobic biodegradation, such as aerobic BOD_5 removal.

(iii) Between two doses (Figure 5.11.C). During this period of the RBF operation, most of the filter bed was highly unsaturated and air can easily diffuse into the filter bed from the surface and down through the filter bed depth. However, the bottom layers of the filter bed are still near or at saturation levels, thereby impeding the movement of air with the effluent stream into the recirculation tank or only limited oxygen can transfer into the recirculation tank. During this period, it is proposed that D.O concentrations were low in the recirculation tank, thereby resulting in an anaerobic environment in which the denitrification process dominated chemical reactions.

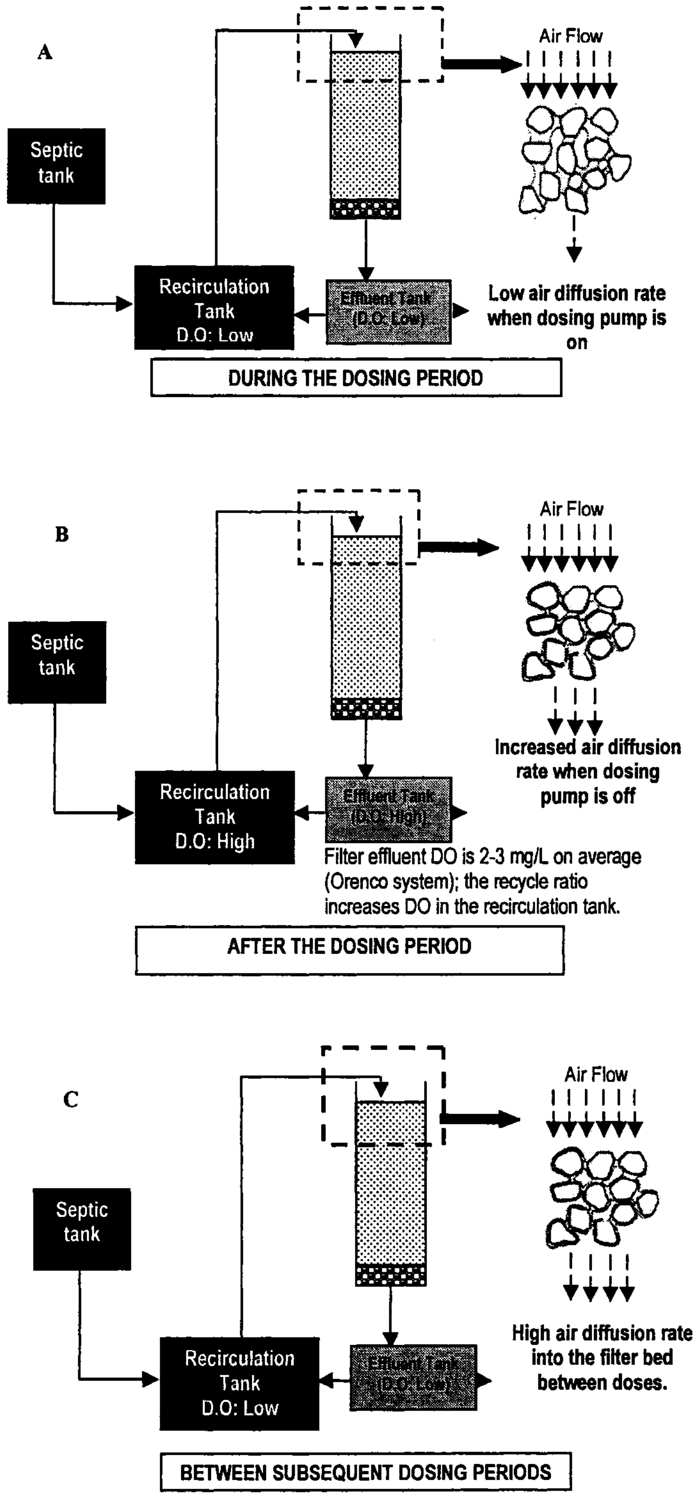


Figure 5.11 Conceptual air flow models for the surface ventilation RBF (A) during the dosing period (B) after the dosing period and (C) between subsequent dosing periods.

<u>Bottom Sidewall Ventilation RBF</u>:

(i) *During the dosing operation (Figure 5.12.A).* The recirculation tank effluent was dosed into the filter bed and air diffusion into the filter bed does not occur due to the covered open surface as well as the saturated (or near saturated) condition in the bottom layer of the filter bed. During this period, limited oxygen is within the filter bed for aerobic biodegradation of organic material to occur.

(ii) After the dosing operation (Figure 5.12.B). The recirculation tank effluent dosing pump is turned off and the wastewater volume begins to drain slowly downward through the filter bed. There is no diffusion of air into the top of the filter bed. However, air is drawn into the bottom layers of the filter bed due to the pressure difference between the inside and outside of the filter bed from the downward flow of the wastewater. At this moment, wastewater begins to drain out of the filter bed and air moved into the effluent and the recirculation tank in a co-current move-mode with water together. Therefore, the D.O in the recirculation tank was high, which could support some aerobic biodegradation, such as aerobic BOD_5 removal.

(iii) Between two doses (Figure 5.12.C). Air diffusion coefficient was low since the bottom layer was near or at saturation condition; and air could not be sucked into the filter bed due to the lack of water movement. During this period, the D.O was low in the recirculation tank, which was more likely to be an anaerobic bioreactor. Therefore, denitrification could occur in the recirculation tank.

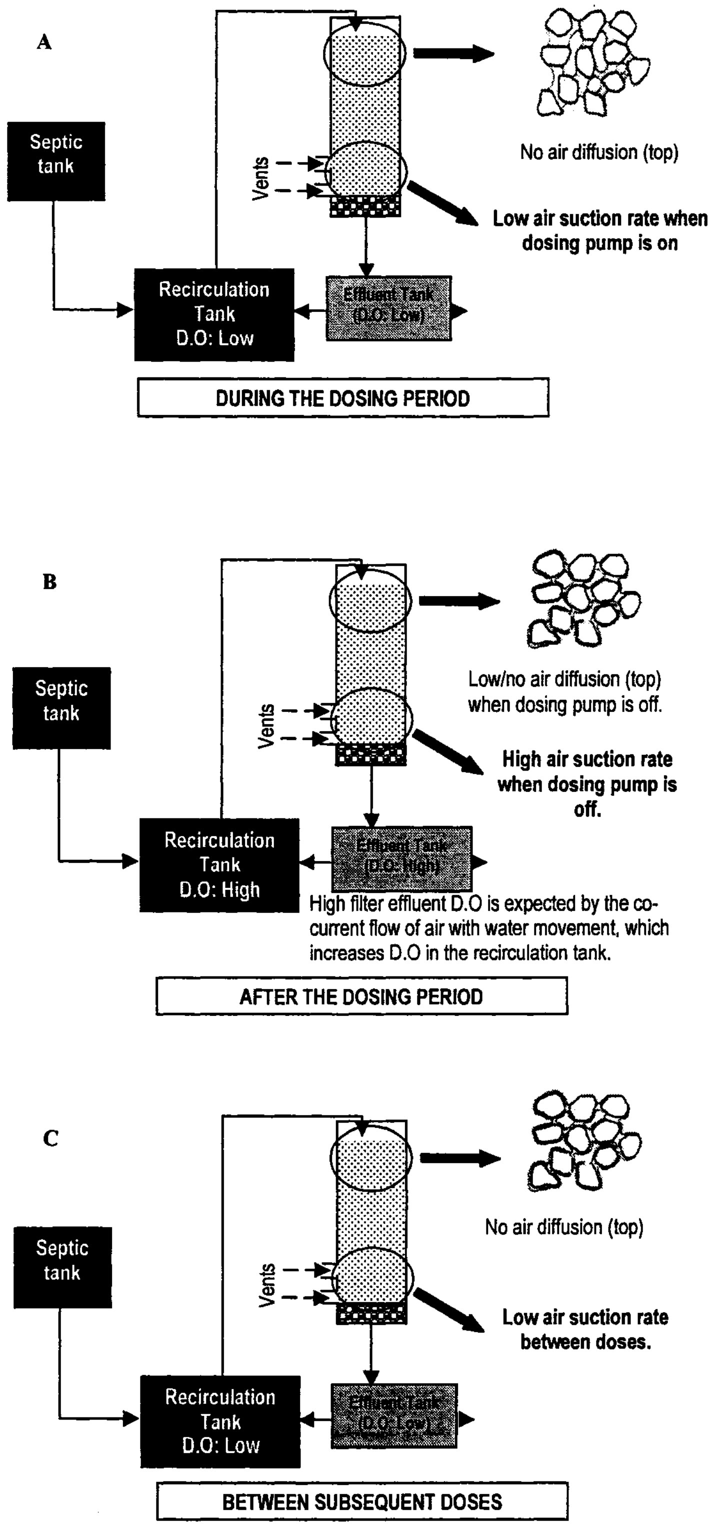


Figure 5.12 Conceptual air flow models for the bottom sidewall ventilation (A) during the dosing operation; (B) after the dosing operation; and (C) between two doses.

Based on the high denitrification process in the recirculation tank of the surface ventilation RBF system, it can be concluded that the oxygen transferred into the effluent and the recirculation tank of the RBF ventilated from the top surface of the filter bed was less than the recirculation tank of the RBF ventilated from the bottom sidewall of the filter bed. Therefore, having ventilation ports at the bottom sidewall of the filter bed actually increased the aerobic biodegradation in the recirculation tank rather than the filter bed.

The conclusion above is helpful to explain the high BOD_5 removal in the combined ventilation RBF (Trial 3) in this chapter, because the aerobic biodegradation of BOD_5 could occur at both of the filter bed and the recirculation tank in Trial 3.

5.5 Summary and Conclusion

This chapter compared three ventilation locations for RBFs: top, bottom sidewall, and top & bottom sidewall of the filter bed. The RBFs effluent water quality including BOD_5, NH_4^+-N, TSS, and fecal coliform was compared among these three types of ventilation designs. The results showed that the RBF ventilated from both the top and the bottom sidewall of the filter bed could improve BOD_5 (the lowest effluent BOD_5 concentration among three RBFs) removal significantly. The results showed that BOD_5 could be nearly removed completely in this design. However, this chapter did not find significant difference among these three RBFs for NH_4^+-N, TSS, and fecal coliform removals at the confidence level of 95%.

This chapter also discussed the water quality in the recirculation tank of RBFs ventilated from the top, and the bottom sidewall of the filter beds. The results showed that both denitrification (TN and NO_3^--N removals) and aerobic BOD_5 removal occurred in the recirculation tank. In addition, the observed water quality showed that the recirculation tank of the RBF ventilated from the top surface of the filter bed was more optimal for denitrification than the recirculation tank of the RBF ventilated from the bottom sidewall of the filter bed.

According to this conclusion and literature results, this chapter provided conceptual air flow modes for RBFs ventilated from the top, and the bottom sidewall of the filter beds. For the surface ventilation RBF, air diffuses into the filter bed after dosing period. With the drainage of water flow, air co-currently moves with water flow into the recirculation tank, where some BOD_5 and NH_4^+-N removals occur. Between two doses, air diffuses into the filter bed to fill the void space. For the bottom sidewall ventilation RBF, no air diffusion occurs from the top surface. After one dose, high oxygen is expected to transfer to the recirculation tank by the co-current follow of air with water movement, which facilitates the BOD_5 and NH_4^+-N removals in the recirculation tank. Between two doses, limited air diffusion occurs through the ventilation holes in the bottom sidewall due to the fully or at near fully saturated condition at the bottom filter bed. These conceptual air flow modes are helpful to explain the different performance, BOD_5 removal in particular, among three RBFs with three different ventilation locations.

6. IMPACT OF FILTER MEDIA ON THE PERFORMANCE FULL-SCALE RECIRCULATING BIOFILTERS FOR TREATING MULTI-RESIDENTIAL WASTEWATER

A version of this chapter has been accepted for publication by *Water Research*

6.1 Abstract

This chapter focused on the comparison of different types of filter media, namely silica sand, crushed glass, peat, and geotextile in a field-scale study as well as investigating the functions of the recirculation tank by the discussion of biological/chemical reaction and the mass balance analysis for various water quality parameters. The field-scale study was conducted in the Municipality of Lunenburg, Nova Scotia, Canada and involved the treatment of domestic wastewater using recirculating biological filters from a small community of ten households. The field-scale plant started to provide service at December, 2003; and the study of Chapter 6 was conducted from June to August, 2004. The average influent BOD_5 and TSS concentrations into the field filter system were 381 ± 64 and 40 ± 23 mg/ L, respectively. The results showed that crushed glass could be an effective RBF medium since the crushed glass filter produced stable effluent BOD_5 and TSS concentrations of less than 20 mg/ L. In addition, geotextile was found to be another successful alternative filter medium with the effluent BOD_5 and TSS of 18 ± 11 and 13 ± 5 mg/ L, respectively, even though the porosity of the geotexitle filter media was as high as 0.90. Peat was not able to provide efficient performance due to poor BOD_5 and NH_4^+-N removals during the field experiments. An analysis of the function of the recirculation tank in this chapter showed that the recirculation tank itself was the main facilitator of TN removal. Finally, this chapter presents the reaction rate coefficients for TN, NH_4^+-N, and BOD_5 removals in the recirculation tank based on mass-balance analysis of the field-scale systems.

6.2 Introduction

Chapters 4 and 5 evaluated the effectiveness of four types of RBF media based on short-term laboratory observations. The evaluation of these media based on the long-term field-scale RBF performance was conducted in this chapter to develop a more improved understanding of the removal mechanisms in RBFs. Therefore, the main objectives of this chapter were:

(i) to evaluate the effectiveness of silica sand, crushed glass, peat and geotextile as the media in RBFs in the removal of organic matter, nutrients, and bacteria from domestic wastewater based on a treatment plant in the Municipality of Lunenburg, NS; and

(ii) to evaluate the function of the recirculation tank by biological and chemical reactions and mass-balance analysis for various water quality components in the recirculation tank for improved understanding of RBF system design.

6.3 Materials and Methods

6.3.1 Description of Field-scale RBF Design

The research was conducted in a small community of ten households in the Municipality of Lunenburg, Nova Scotia, Canada. Since most of the homes in this community had either non-existent or malfunctioning onsite sewage disposal systems, recirculating biofilters were designed as an alternative technology to treat domestic wastewater from this community.

The design was based on a wastewater flow rate of 10,000 L/ day and a loading rate of 0.16 m/ day. An additional redundant sand cell was provided to ensure full available capacity should one cell be required to be offline for an extended period of time. Therefore, the system was divided into five cells, with dimensions of 6 m by 1.8 m each, for a total filtration area of 54 m^2. Two of the cells contained sand while the remaining three cells contained crushed glass, peat and geotextile media, respectively (Figure 7.1). Each cell had its own shut-off valves, flow meters, and effluent sampling ports so that adjustments could be made to the loading rate of each cell, irrespective of the operation of the other cells.

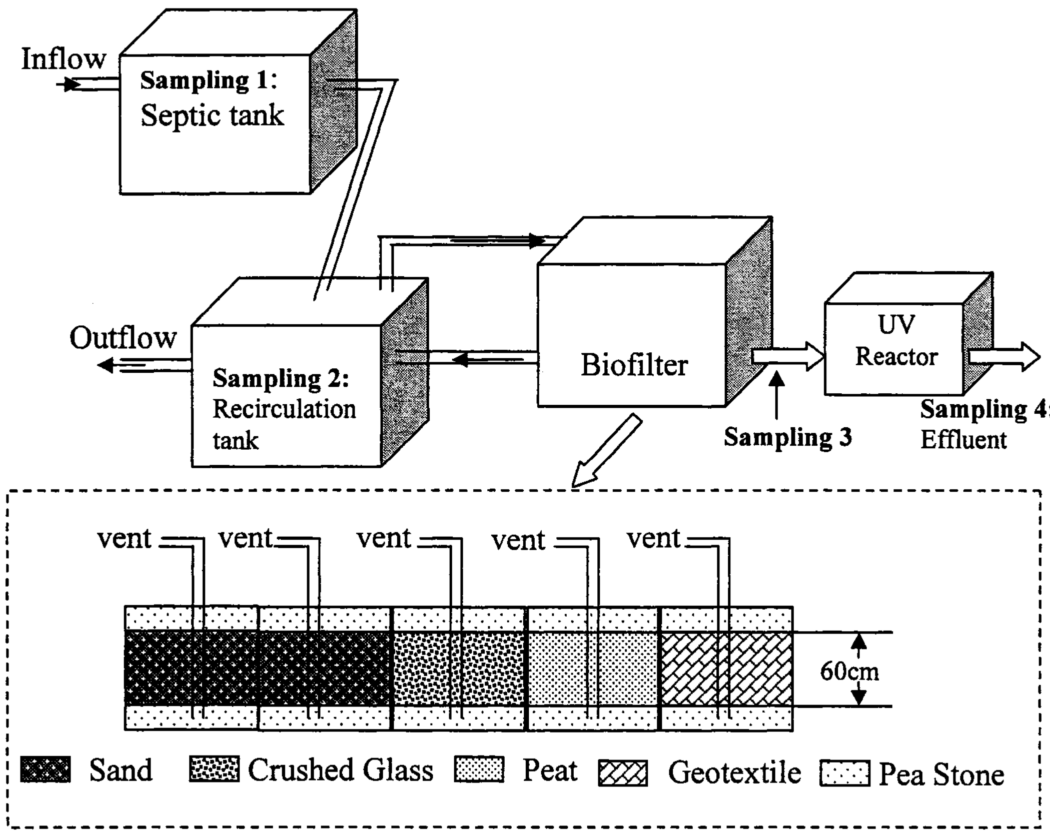


Figure 6.1 Field-Scale RBF Design.

Septic tanks were used to collect sewage wastewater from each home in the community. The effluent from the septic tanks was then discharged into an 11,350 L pre-cast recirculation tank. Screened submersible duplex pumping systems directed the effluent to five recirculating biological filter (RBF) cells based on timed dosing such that an even loading was achieved over the diurnal cycle. The dosing frequency of the field-scale RBFs was 206 times per day, with a 3.5-minute running time for each dose and 3.5-minute off-time between doses. For each dose, a portion of the filtered effluent (80 %) was pumped back to the recirculating tank for re-dosing into the filter, and the remaining 20% was passed through a UV unit for inactivation of pathogenic microorganisms.

The distribution network for the RBF design was constructed using PVC pipes packed in pea gravel. Each cell contained a six-zone pressure operated distributing valve that fed the manifold pipes (38-mm diameter). The manifold pipes distributed the effluent through 15 lateral pipes (25-mm diameter.) that had orifices (3-mm diameter) located at 600 mm spacing intervals. The total number of orifices was 150 for the manifold with each orifice delivering less than 5.7 L/ dose. The distribution piping was designed with the orifices pointed down to prevent freezing under winter conditions. The effluent percolated through the media, and was collected by a PVC pipe (100-mm) underdrain network. The underdrain network collected and conveyed the effluent back to the recirculation tank.

The top of the filter was exposed to the atmosphere to allow for air flow into the top of the filter. PVC vents were provided to ensure air could get to the bottom of the filter. In other words, the field-scale RBFs in this chapter were ventilated from both the top and bottom of the filter bed, as one of the ventilation methods described in Chapter 5.

The 20% of the effluent that was not recirculated was directed by gravity to an ultraviolet (UV) disinfecton system housed within a precast concrete chamber. The system was designed to provide 99.9% reduction in bacteria and a bacterial fecal colony count of less than 200 per 100 ml in effluent suspended solids concentrations of less than 10 mg/ L. The local discharge requirements were 20 mg/ L for both BOD_5 and TSS.

6.3.2 Data collection and analysis

The field-scale RBFs were commissioned in October, 2003 to provide service for this small community in Lunenburg, N.S. The present study was conducted after the system had been in continuous operation for nine months (June to August, 2004). Samples were collected and stored by treatment plant operational staff; and stored samples were shipped to Dalhousie University for measurement. During the sampling period, the local temperature was between 15 ~ 25°C. Room temperature was maintained when the samples were measured in the laboratory at Dalhousie University. As presented in Figure 6.1, samples were collected from the septic tank, filter effluents, recirculating tank and the UV system effluent. BOD_5, TN, NH_4^+-N, pH, fecal coliform, TSS and TP concentrations were measured using methods described in Chapter 3.

6.4 Results and Discussion

6.4.1 Septic Tank Effluent

Table 6.1 presents the wastewater quality characteristics from samples taken from the septic tank pump chambers. Due to the shortage of water supply in this community and wastewater mainly coming from bathroom and kitchen, the average septic tank BOD_5 measured in samples (381 ± 64 mg/ L) was higher than BOD_5 concentrations reported in other septic tank studies (Babcock et al., 2004; Spychata and Blazejewski, 2003; and Hellstrom and Jonsson, 2003).

Table 6.1 Septic tank effluent data

	BOD_5 (mg/L)	TN (mg/L)	NH_4^+-N (mg/L)	TP (mg/L)	TSS (mg/L)	pH	Fecal coliform (col./100mL)
Median	381	149.1	103.8	8.5	40	7.3	1.0×10^7
Minimum	173	90.0	56.8	6.9	18	6.6	1.0×10^5
Maximum	479	265.0	176.0	13.0	85	8.4	6.0×10^8
Standard deviation	64	62.4	39.9	1.8	23	0.7	1.6×10^8
Number of samples	21	19	17	8	13	22	19

6.4.2 Recirculating Biofilters

As described previously in Chapter 5, treatment in the recirculating media filter is brought about by complex physical, chemical and biological transformations. The removal of soluble BOD_5 and nitrification, the conversion of ammonium to nitrate, is performed by microorganisms present in the media bed and occurs under aerobic conditions. Suspended solids are removed principally by mechanical straining, straining due to chance contact with media particles and sedimentation. Phosphorous is removed by adsorption to the filter media.

This study was able to collect individual effluent after biofiltration for each RBF cell as shown in Figure 6.1. However, the four types of RBFs effluents were pumped into a common recirculation tank. Therefore, the actual water quality dosed into each filter bed (or water quality leaving the common recirculation tank) was similar.

Organic Matter Removal. The filter effluents were observed to have good clarity and no odor. As presented in Figure 6.2, the overall average effluent BOD_5 measurements during the field trials were 15 ± 10, 11 ± 7, 39 ± 29, and 18 ± 11 mg/ L for the sand, crushed glass, peat and geo-textile filters, respectively.

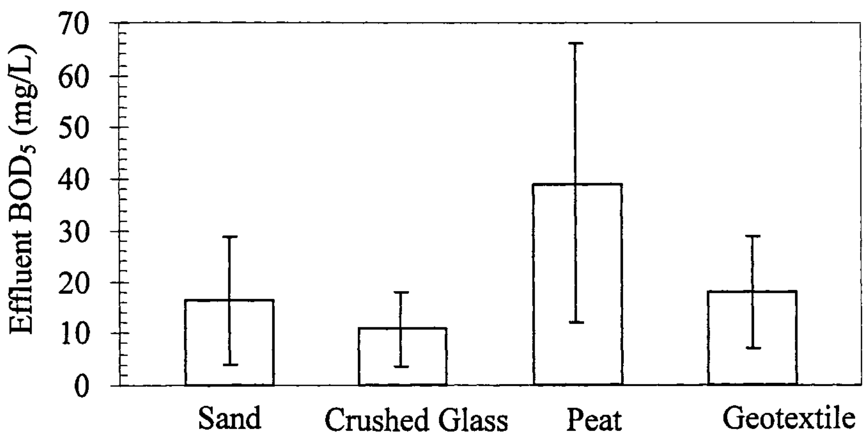
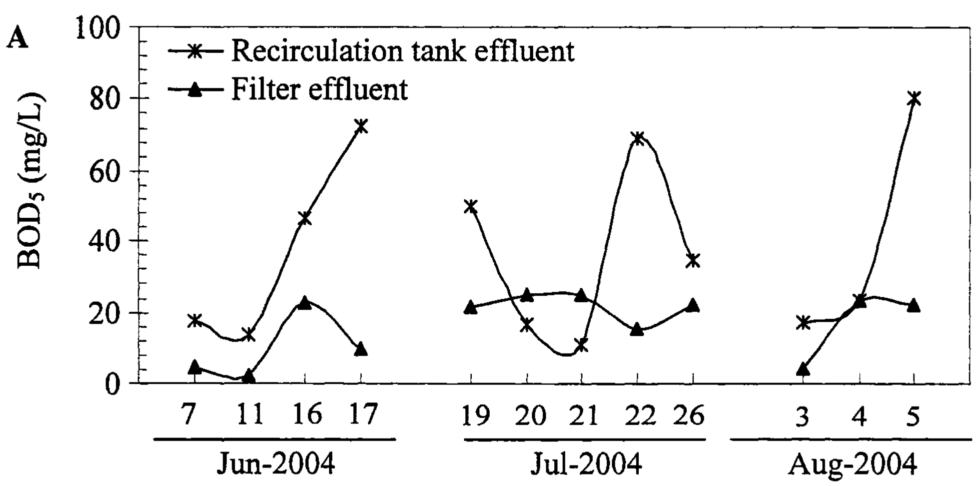
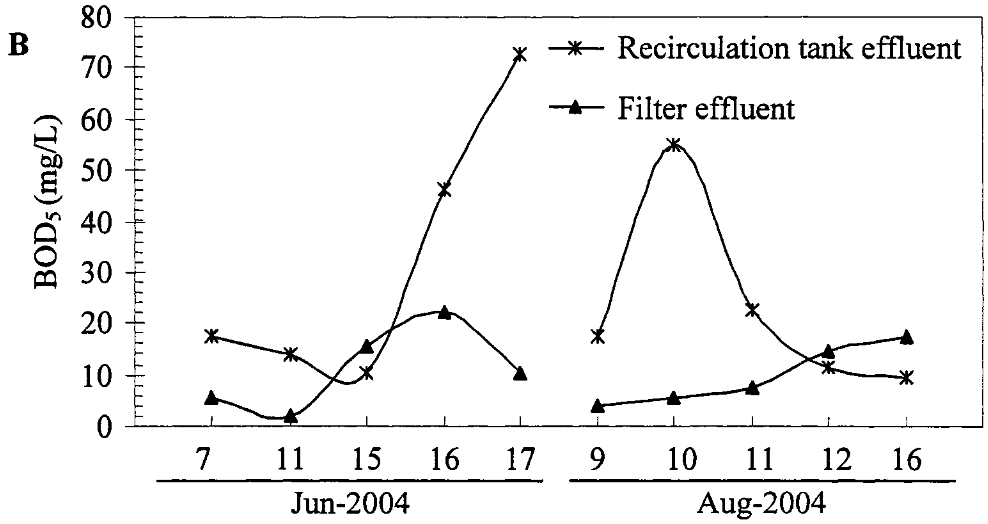


Figure 6.2 Effluent BOD_5 concentration following biological filtration (Error bars indicate standard deviation from mean)

The crushed glass RBF cell produced a more stable effluent in terms of BOD_5 concentrations despite fairly large fluctuations in the septic tank BOD_5. As shown in Table 6.1, the influent BOD_5 ranged from 173 to 479 mg/ L over the course of study. The results of an ANOVA showed that there was no significant difference between the sand and the crushed glass filters in terms of the BOD_5 percent removal, as shown in Appendix: J – ANOVA – Field Scale – BOD_5. This conclusion is consistent with the result from bench-scale test in Chapter 4 that crushed glass and sand performed similarly as a medium for RBFs under identical design. In addition, the results of this field-study demonstrate that geotextile could be an effective filter medium for RBFs, since the average effluent BOD_5 concentration after geotextile biofiltration was less than 20 mg/L, which is in compliance with local effluent regulations (e.g., BOD_5 < 20 mg/ L). Again, this finding is consistent with the conclusion from the bench-scale studies in Chapter 4 that geotextile was able to produce an effluent BOD_5 concentration less than 20 mg/ L. Finally, this chapter found that the average effluent BOD_5 concentration from the peat filter was 39 mg/ L with a maximum BOD_5 concentration of 85 mg/ L. In contrast to the bench-scale results presented in Chapter 4, the effluent of field-scale peat filter was observed to have no color and good clarity.

To investigate the aerobic BOD_5 biodegradation within the filter bed, this chapter compared the actual recorded BOD_5 concentrations entering the filter bed (recirculation tank effluent) to the actual recorded BOD_5 concentrations leaving the filter bed (filter effluent). Figure 6.3 shows that aerobic BOD_5 removal occurred in each of the field-scale RBFs during the study. The average BOD_5 removal in the sand RBF was 58 %, with average influent and effluent BOD_5 concentrations of 38 and 16 mg/ L, respectively (Figure 6.3.A). Figure 6.3.B demonstrates that a 64 % reduction of BOD_5 was achieved in the crushed glass filter bed, with average influent and effluent BOD_5 of 28 and 10 mg/ L, respectively. The average BOD_5 removal in the peat filter bed was 35% with influent and effluent BOD_5 of 51 and 33 mg/ L, respectively (Figure 6.3.C) The average BOD_5 removal in the geotextile filter bed was 39% with influent and effluent BOD_5 of 31 and 19 mg/ L, respectively (Figure 6.3.D).




B
80
70
60
50
40
30
20
10
0
BOD5 (mg/L)
Recirculation tank effluent
Filter effluent
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11
15
16
17
9
10
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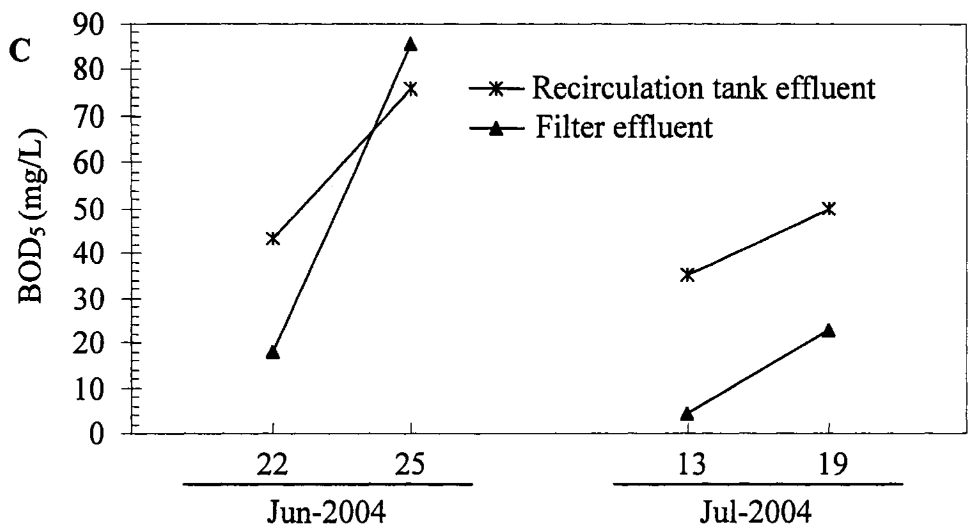
12
16
Jun-2004
Aug-2004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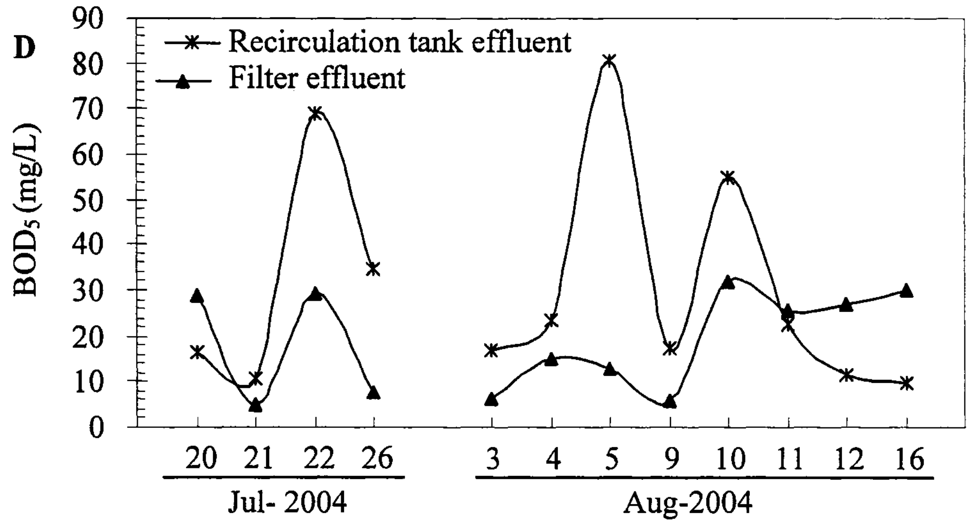


Figure 6.3 Comparison between BOD_5 entering and leaving the filter bed: (A) sand; (B) crushed glass; (C) peat; and (D) geotextile.

TSS removal. Figure 6.4 shows the average total suspended solids (TSS) concentrations from samples taken from the four RBF cells during the field study. The sand filter showed an unstable effluent TSS concentration of 33 ± 30 mg/ L. As presented in Figure 6.4, the results of this study showed that the sand filter did not achieve removal of TSS, since the sand filtration effluent TSS was consistently higher than the recirculation tank effluent (the actual influent to the filter bed).

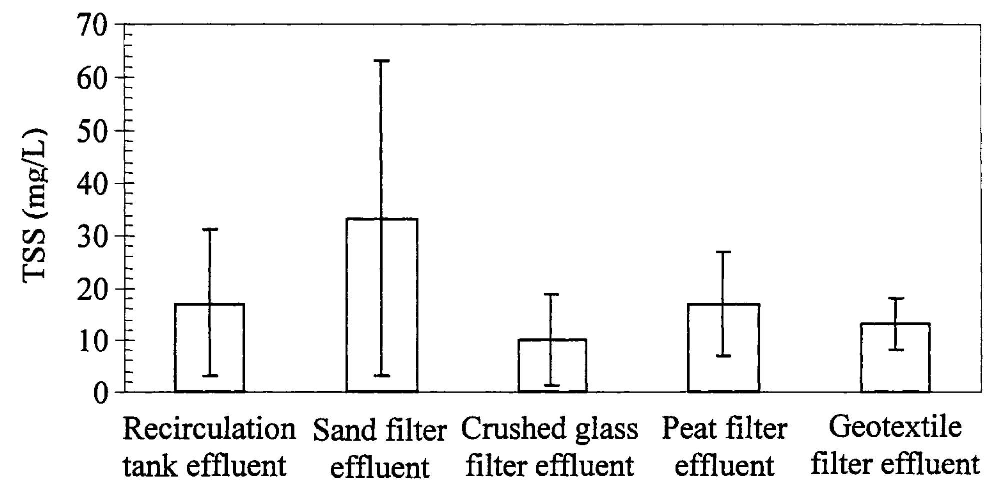


Figure 6.4 Effluent TSS concentration following biological filtration (Error bars indicate standard deviation from mean)

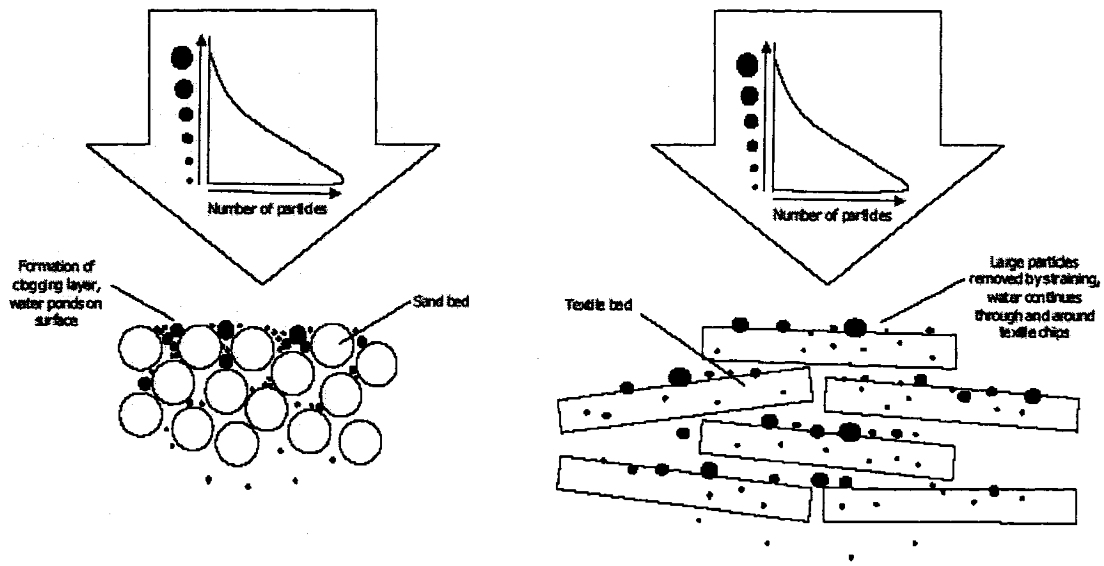
The average TSS removal after the sand filtration (compare to the septic tank effluent TSS) was 31%, which was much lower than the results of Babcock et al. (2004), Louden et al. (1985), Piluk and Peters (1994), Ronayne et al. (1982), Roy and Dube (1994), and Owen and Bobb (1994). The results of those studies showed 89.7, 85.7, 89.3, 97.3, 96.1 and 83.3 % removal of TSS, respectively. Loudon (1985) reported on a study of a recirculating sand filter used for residential wastewater treatment in Gloucester, MA, US. That achieved an average effluent TSS concentration of 2 mg/ L. However, another study which evaluated a recirculating sand filter for residential wastewater treatment in Minnesota, US, produced an effluent TSS concentration of 23 mg/ L (Christopherson et al., 2001), which was quite close to the results of the present study.

As presented in Figure 6.4, the crushed glass filter showed higher and more stable levels of TSS removal as compared to the sand filter, with an average filter effluent TSS concentration of 10 ± 9 mg/ L. More than 40% of the recirculation tank effluent TSS was removed by the crushed glass filter bed. CWC (1997) and Elliott (2001b) reported on studies of two recirculating crushed glass filters for residential wastewater treatment in King Co., WA and Oswego, NY, respectively. The average effluent TSS concentrations were 4 and 3 mg/ L for CWC (1997) and Elliott (2001b), respectively. These TSS removal results are lower than the TSS removal achieved in the current study. However, the crushed glass recirculating filter in this study produced an average effluent TSS concentration less than 10 mg/ L, which is in compliance with local discharge regulations (e.g., TSS < 20 mg/ L).

The peat filtration average effluent TSS was 17 ± 10 mg/ L, which also met the local regulation of effluent TSS less than 20 mg/ L. However, Figure 6.4 indicates that the peat filter bed did not provide effective TSS removal due to the similar average TSS concentrations between recirculation tank and peat filter bed effluents. Currently, peat is widely applied in single-pass biofilter systems, such as the ECO-PURE® peat filter. Leverenz et al. (2001) reported on three single-pass peat biofilters used for residential wastewater treatment in Maine, U.S. Average effluent TSS concentrations reported in their study with this system was between 9 to 16 mg/ L, which is consistent with the results of the current study.

An important finding of this study was that the geotextile recirculating filter was able to achieve high levels of TSS removal with average filter effluent values of 13 ± 5 mg/ L, even though the porosity of the geotexitle filter was as high as 0.90. The geotexitle filtration effluent TSS was slightly lower than the recirculation tank effluent dosed into the filter bed. Leverenz et al. (2000) investigated the effectiveness of geotextile chips as a RBF medium. In that study, three different filter bed configurations were investigated, including hanging sheet, single layer, and three layers. The average effluent TSS concentration from these three configurations under a hydraulic loading rate (HLR) of 0.41 $m^3/m^2/day$ was 1 mg/ L. The porosity of geotextile chips in that study was 0.95, which is very close to the porosity of the geotextile filter used in the present study. The results of their investigation and the current study demonstrate that both geotextile fiber and geotextile chips could be effective RBF media for TSS removal, although their porosity values were much higher than traditional filter media such as sand. The porosity of both the sand and crushed glass filter in the current study was 0.36.

The findings of the present study are consistent with the conceptual model of TSS removal by geotextile chips as presented in Figure 6.5 (Leverenz H. et al., 2000). That study proposed that the surface area of geotextile available for large particle filtration is much greater than that of a sand bed. The increased surface area available for filtration makes it possible to treat an equivalent volume of wastewater in a smaller area, as compared to sand filters. In another words, a RBF with geotexitle media with a large surface area was expected to filter more particles than a sand-based filter.



A B

Figure 6.5 Conceptual model of particle removal in (A) sand bed and (B) textile fabric bed (source: Leverenz et al., 2000).

NH_4^+-N Removal. As presented in Chapter 4, results of bench-scale RBFs studies found that nearly complete ammonium (NH_4^+-N) removal could be achieved in RBF design. As presented in Chapter 5, average effluent NH_4^+-N concentrations were found to be less than 1.0 mg/ L in three pilot-scale RBFs ventilated at three different locations. In addition, USEPA (2002) reported that recirculating sand filters are capable of almost complete removal of NH_4^+-N. However, other studies have reported on results that showed that packed bed filters produce high effluent NH_4^+-N concentrations, such as greater than 10 mg/ L (Leverenz et al., 2001). Elliott (2001a) reported on the operation and performance of a recirculating crushed glass filter serving a small community in Oswego, NY. In that study, the average effluent NH_4^+-N was found to be 4.1 mg/ L. Jantrania et al. (1998) reported on a study evaluating a crushed glass filter treating residential in Gloucester, MA. In that study, the average NH_4^+-N concentration in the filter effluent was reported to be 17.5 mg/L. Given the highly variable filter effluent NH_4^+-N concentrations reported in literature, this study investigated the effluent NH_4^+-N concentrations during the field-scale RBF study.

The average sand filtration effluent NH_4^+-N was 0.7 ± 0.4 mg/ L (Figure 6.6), which provided more than 99% removal of NH_4^+-N calculated based on the septic tank to the filter effluent. The average recirculation tank effluent NH_4^+-N was 16.7 ± 16.3 mg/ L which resulted in an average 96 % removal of NH_4^+-N by the sand filter bed based on the recirculation tank to the filter effluents.

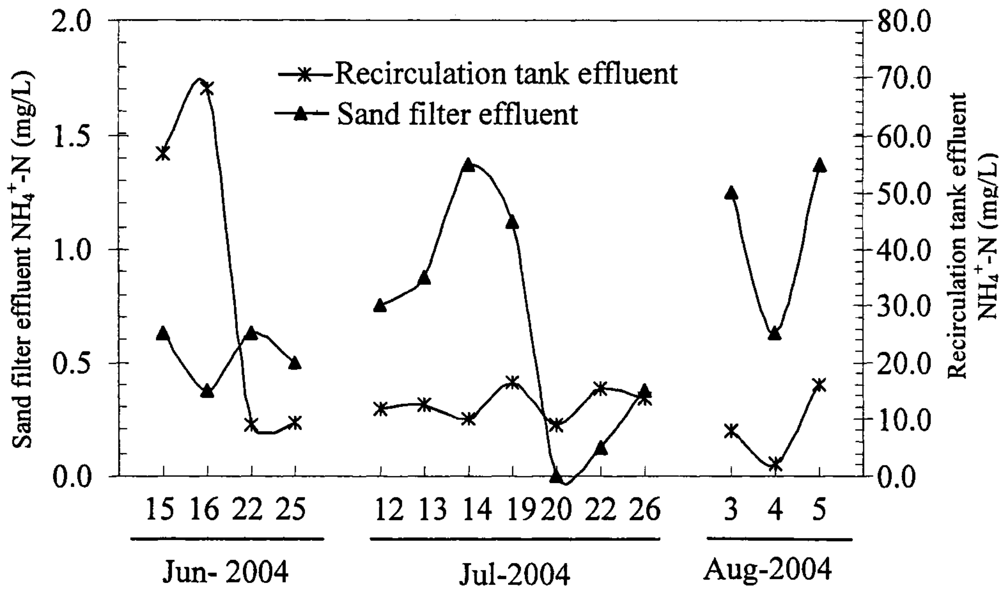


Figure 6. 6 Comparison of recirculation tank effluent and sand filter effluent NH_4^+-N

As presented in Figure 6.6, the average crushed glass filter effluent NH_4^+-N concentration was found to be 1.1 ± 0.6 mg/ L which resulted in 99 % reduction in ammonium when calculated using the average septic tank effluent NH_4^+-N concentration. Based on the average recirculation tank effluent NH_4^+-N concentration, the crushed glass filter reduced NH_4^+-N by 93 %.

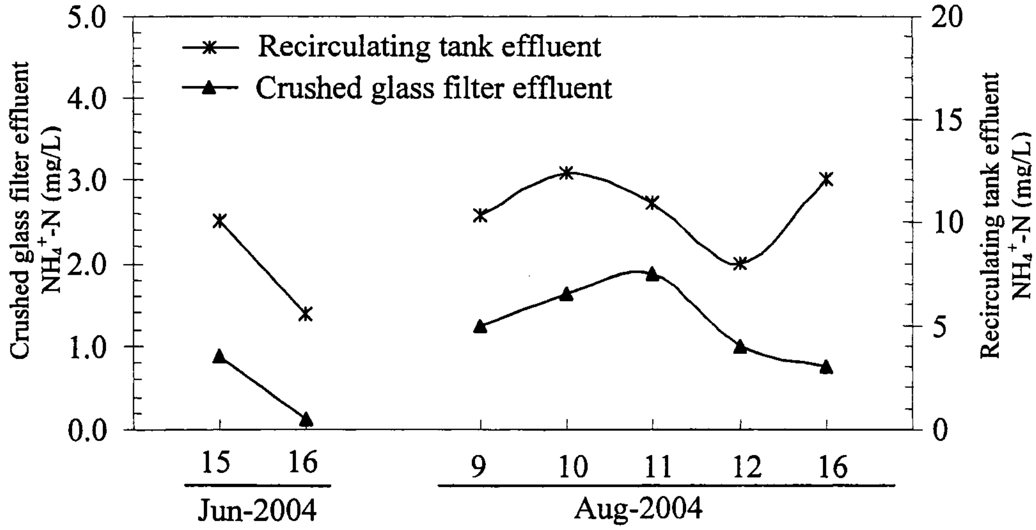


Figure 6.7 Comparison of recirculation tank effluent and crushed glass filter effluent NH_4^+-N

Figure 6.8 shows the average recirculation tank effluent and peat filter effluent NH_4^+-N concentrations. In contrast to the performance of the sand and crushed glass filters, the peat filter effluent NH_4^+-N was higher (17.4 ± 2.9 mg/ L) than the recirculation tank effluent NH_4^+-N (11.4 ± 2.8 mg/ L). These results demonstrate that there was no removal of NH_4^+-N in the peat filter bed during the field-study. The high filter effluent NH_4^+-N concentrations observed in the field-scale peat RBF are consistent with results of the bench-scale study presented in Chapter 4.

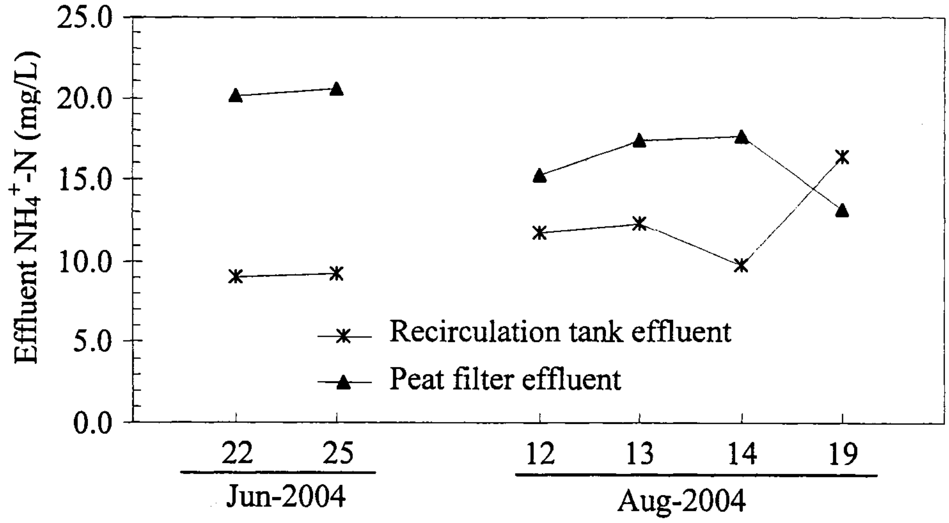


Figure 6.8 Comparison of recirculation tank effluent and peat filter effluent NH_4^+-N

Figure 6.9 shows the average recirculation tank effluent and geotextile filter effluent NH_4^+-N concentrations measured in samples taken during the field study. The geotextile filter effluent NH_4^+-N concentration averaged 4.5 ± 1.1 mg/ L through the study. Although the removal compared with septic tank was 96%, the geotextile filter did not produce an effluent NH_4^+-N concentration less than 1.0 mg/ L on average, which is the recommendation of the Canadian Water Quality Guideline. The reduction in NH_4^+-N concentration within the geotextile filter bed was only 73 %, which was lower than the removals achieved in the sand and crushed glass filters. The average geotextile filter effluent NH_4^+-N concentration measured in the field-scale system was also higher than the bench-scale geotextile filter effluent NH_4^+-N concentration previously discussed in Chapter 4. In the bench-scale studies, an effluent NH_4^+-N concentration of 5.5 mg/ L was achieved. In addition, Leverenz et al. (2000) reported that the effluent NH_4^+-N concentration from recirculation geotextile chips filter was less than 1.0 mg/ L, on average. The results of current study are not consistent with their observation.

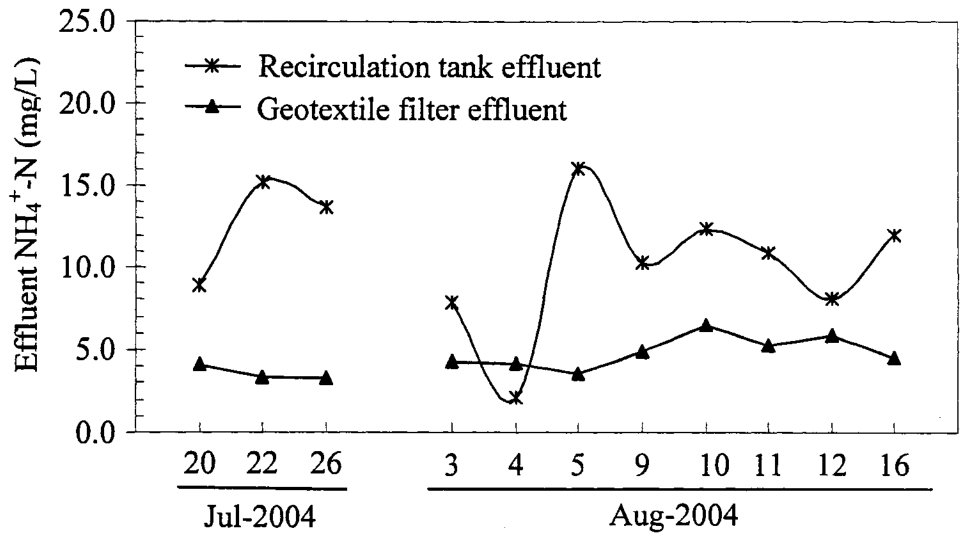


Figure 6.9 Comparison of recirculation tank effluent and geotextile filter effluent NH_4^+-N

The main mechanism of NH_4^+-N removal within the filter bed in the field studies was assumed to be the nitrification process. Although the effluent nitrate (NO_3^--N) was not measured, the results of the pilot-scale RBF studies (Chapter 5) and literature have demonstrated that nitrification is the main bio-reaction for NH_4^+-N removal within RBF filter beds. As described in Chapter 5, the optimal pH for nitrification is in the range of 7.2 to 9.0 (Metcalf and Eddy, 2003). Figure 6.10 shows that the average pH values of the recirculation tank and the sand filter effluent were 6.5 ± 0.4 and 6.2 ± 0.3, respectively. The pH measured in the crushed glass, geotextile and peat RBF filter effluents were also less than 7.0 through the study. Therefore, the pH of the influent to the RBF filters (or recirculation tank effluent) and the effluent leaving the three field-scale RBF filter beds was not the optimal value for nitrification. However, more than 96% of the NH_4^+-N removal was achieved within the sand filter bed. This conclusion was consistent with the result of Mosley (2001) that nitrification could be driven to completion even under somewhat adverse or less optimal pH conditions. Mosley (2001) reported that the pH level in the effluent was low (i.e., 3.5 to 3.9) and the lowering of pH in the sand filter effluent did not prevent nitrification from occurring within the sand filter bed.

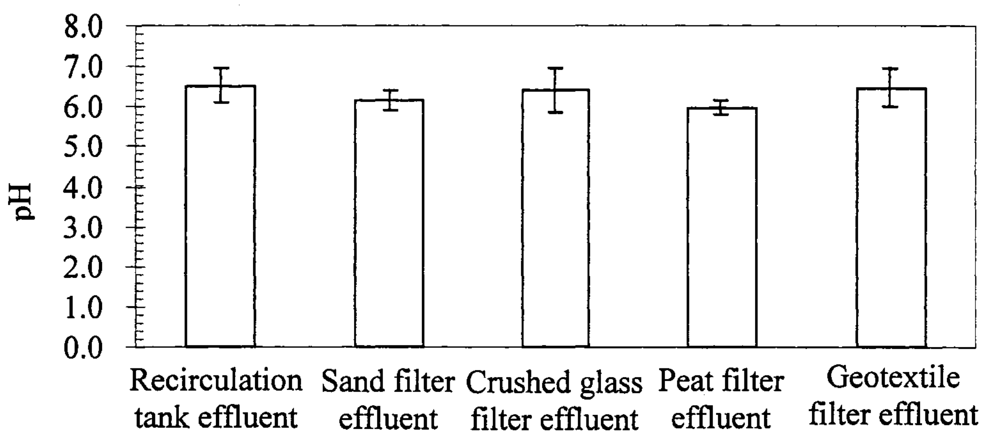


Figure 6.10 Effluent pH of recirculation tank and RBF filter effluents (Error bars indicate standard deviation from mean)

Similar to the sand RBF performance, the crushed glass filter bed also removed NH_4^+-N effectively (i.e., 93 % removal) under the low pH (i.e., 6.2) environment. The average peat filter effluent pH was 6.0 ± 0.2, which was very close to the other filter effluents. However, the field-scale peat RBF effluent pH was found to be higher than effluent pH measured in the bench-scale RBF study presented in Chapter 4. The average peat filter effluent pH was found to be 4.0 in the bench-scale test during 6 weeks of operation. A similar peat was used as the filter medium for both bench- and field-scale RBFs. Patterson et al. (2001) and Talbot et al. (1996) reported that single-pass peat filters removed NH_4^+-N in the septic tank effluent by 94.5 % and 60 %, respectively. However, poor NH_4^+-N removal with peat filters has also been reported in literature. Leverenz et al. (2001) reported effluent NH_4^+-N concentrations of 10.4 and 17.7 mg/ L for two peat filters treating residential wastewater in Maine. Leverenz et al. (2001) reported on two peat filters treating residential wastewater that produced an effluent NH_4^+-N concentration of 25.0 and 19.0 mg/ L under the single- and multi-pass operational modes, respectively. Therefore, the results of the current study conducted at both bench- and field-scale are consistent with the results of Leverenz et al (2001) and demonstrate that NH_4^+-N removal in peat RBFs may be more difficult than in RBFs loaded with alternative media.

TP Removal. USEPA (2002) reported that adsorption was the main mechanism for total phosphorus (TP) removal in RBFs. However, the adsorption capacity within the filter has been shown to decrease over the operation time of the RBF. The present study was conducted after the field-scale RBF system had been operational for nine months. As presented in Figure 6.11, all of the RBFs did not achieve significant TP removal during the test period. The average TP concentration in the septic tank effluent and recirculation tank effluent (influent to the filters) during the field experiments were 9.1±1.8 and 11.1±3.7 mg/ L, respectively. Average effluent TP concentrations of 8.6 ± 1.6, 11.6 ± 5.4, 9.3 ± 0.4, and 12.6 ± 3.6mg/ L were measured from the sand, crushed glass, peat, and geotextile filters effluent samples, respectively. Both the crushed glass and geotextile RBF filters produced higher TP concentrations in the filter effluent as compared to the septic tank effluent TP concentrations. Babcock et al. (2004) reported on a sand RBF system that achieved a reduction in TP from 4.6 mg/ L in the septic tank to 3.8 mg/ L in the filter effluent, or 17 % removal. The results of that study and those reported in the present study are consistent with the conclusion of USEPA (2002) that phosphorus removal decreases from a high percentage to about 20 to 30% after the exchange capacity of the media becomes exhausted. This conclusion is also consistent with the results of the bench-scale studies presented in Chapter 4. The bench-scale studies demonstrated that the geotextile filter could provide the highest TP removal as compared to the other filter media evaluated. However, the low TP removal from the geotextile in the field-scale study suggests that the adsorption capacity of geotextile filter became exhausted after 9-month operation.

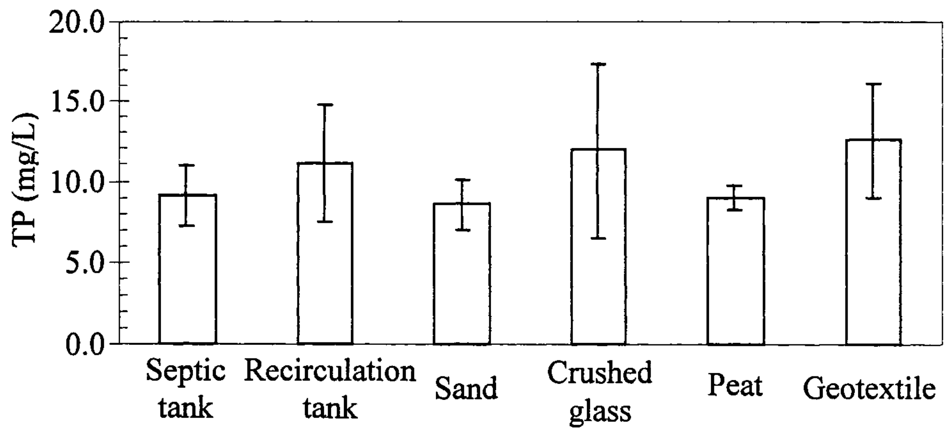


Figure 6.11 Effluent TP of septic tank, recirculation tank, and filtration effluents (error bars indicate standard deviation from mean)

Fecal Coliform Removal. Fecal coliform is reduced by attachment and die off of the bacteria to the media as the wastewater flows through the porous media. Fecal coliform counts in the filter effluents varied significantly among the four field RBF systems evaluated in this study. The sand filter effluent had the highest fecal coliform count with $1.5 \times 10^7 \pm 4.0 \times 10^6$ CFU/ 100mL. The peat filter showed the best fecal coliform removal with an average effluent count of 1.0×10^4 CFU/ 100mL. The average effluent fecal coliform count of the crushed glass and geotextile filters were $8.6 \times 10^4 \pm 6.1 \times 10^4$ and $9.6 \times 10^5 \pm 4.5 \times 10^5$ CFU/ 100mL, respectively. Using the average septic tank effluent fecal coliform count ($1.6 \times 10^8 \pm 7.9 \times 10^7$ CFU/ 100mL), the log reductions of fecal coliform in this study were calculated to be 1.3, 3.1, 3.9 and 2.2 for sand, crushed glass, peat, and geotextile RBFs, respectively. The results of this study are similar to values reported in literature of 99 to 99.9 % reductions in fecal coliform counts (USEPA, 2002). Other studies have shown that recirculating sand filters did not achieve high fecal coliform log reductions, with reported reductions ranging from 1.5 to 1.7-log (Ronayne et al. 1982, Roy and Dube, 1994). CWC (1997) reported that the treatment of domestic wastewater with two recirculating crushed glass filters resulted in fecal coliform log-reductions of 2.7 and 3.3 in King Co., WA and Ronald, WA, respectively. These results are consistent with the high log-reductions of fecal coliform achieved in the field-scale crushed glass filter of the present study (e.g., 3.1 log-reductions). Peat has been shown to be an effective biofilter material particularly for the removal of fecal coliform due to its characteristics. Leverenz et al. (2001) reported on two peat biofilter treating residential wastewater in Maine that showed fecal coliform log-reductions of 6.8 and 6.4. However, the two biofilters reported in that study were single pass biofilters. As discussed in Chapter 2, single pass biofilters can provide higher fecal coliform log-reductions than multiple pass biofilters, probably because fecal coliform get mutant after multiple pass through filter beds. This conclusion explains the fact that the fecal coliform log-reductions found in the present study were lower than those reported by Leverenz et al. (2001). However, the field-scale peat filter still produced the lowest and the most stable effluent fecal coliform among the four types of filters in the present study.

6.4.3 Chemical Reactions within the Recirculation Tank

The field-scale RBFs were ventilated from both the top and the bottom of the filter beds, which is consistent with one of the ventilation modes investigated in the pilot-scale studies presented in Chapter 5. Therefore, this chapter investigated the function of the recirculation tank based on three of the measured parameters in the field-scale RBF systems, including BOD_5, NH_4^+-N, and TN. An analysis of the concentrations of these test parameters in the RBF systems suggest that a combination of biological and chemical reactions were occurring in the recirculation tank, specifically TN removal, aerobic removal of BOD_5, and nitrification.

TN Removal

A projected TN in the recirculation tank was calculated using Equation 5.1 in Chapter 5. However, the projected TN and other water quality parameters were determined by the combined filter effluent and the septic tank effluent. In this chapter, the combined filter effluent water quality was calculated by the average value of four types of RBFs effluents water quality, since all the RBFs cells share a common recirculation tank. For example, the effluent TN concentrations of four types of RBFs were 38.1, 32.2, 46.1, and 27.7 mg/ L for the sand, crushed glass, peat, and geotextile, respectively. Therefore, the combined filtration effluent TN was 36.0 mg/ L

((38.1+32.2+46.1+27.7)/4 = 36.0). Therefore, the projected TN concentration in the recirculation tank was determined by both combined filtration effluent TN and the septic tank effluent TN ((36.0 × 4 + 149.1) / 5 = 58.6 mg/ L). As presented in Figure 6.12, these data suggest that TN was removed in the recirculation tank. The average projected TN in the recirculation tank was calculated to be 53.2 ± 2.6 mg/ L and the measured TN in the recirculation tank was found to be 41.9 ± 7.3 mg/L. Based on these data, approximately 21 % of the TN was removed in the recirculation tank.

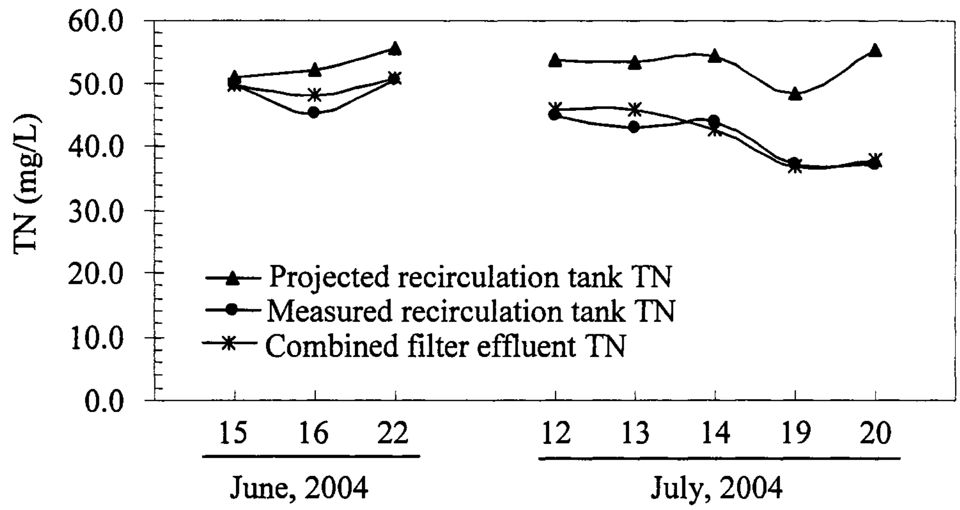


Figure 6.12 Comparison of projected TN in the recirculation tank and measured TN in the filter effluent

Figure 6.12 also shows the TN concentration in the combined filter effluents stream. The TN concentrations measured in the combined filter stream and the recirculation tank effluents were very close in value, indicating that TN removal did not occur in the filter bed. Collectively, the data shown in Figure 6.12 demonstrates that the TN removal was driven by reactions occurring in the recirculation tank, since limited TN removal was observed in the filter beds. The overall TN removal of the field-scale RBFs in the present study was found to be 56 %using the septic tank effluent average TN concentration of 95.6 ± 36.9 mg/ L. The overall TN reduction found in the present study is consistent with Elliott (2001b) and Loudon (1985) which found TN removals of 55 and 53 %, respectively, for RBF systems treating the residential wastewater. The results of this study and analysis confirm the advantage of multiple pass packed bed filters, or RBFs, over single pass packed bed filters. Specifically, the presence of the recirculation tank in RBF design allows for TN removal to occur in contrast to single-pass packed bed filters which are often used for smaller applications where nitrogen removal is not required (USEPA, 2002).

NH_4^+-N Removal

Figure 6.13 shows the projected and actual measured NH_4^+-N in the recirculation tank and the measured combined filter effluents NH_4^+-N concentration. Projected NH_4^+-N concentrations were calculated using Equation 5.1. The actual measured recirculation tank effluent NH_4^+-N was determined by recording NH_4^+-N in the recirculation tank effluent samples. The combined filter effluents NH_4^+-N was determined by the average NH_4^+-N concentrations of four types of RBFs effluents, which were measured separately.

Figure 6.13 shows that an overall reduction in NH_4^+-N concentrations occurred between the recirculation tank effluent (influent to RBFs) and the combined filter effluent stream based on field measurements. Specifically, the actual NH_4^+-N concentration in the recirculation tank effluent stream averaged 10.6 ± 2.8 mg/ L while the combined filter effluent NH_4^+-N concentrations averaged 3.4 ± 1.1 mg/ L during the field experiments, although peat filter bed did not remove NH_4^+-N effectively as shown in Figure 6.8. Figure 6.13 also demonstrates that there was a difference between projected and actual measured NH_4^+-N concentrations in the recirculation tank. Based on these data, it can be concluded that NH_4^+-N removal occurred in the recirculation tank. Although the field RBFs study in this chapter did not quantify NO_3^--N in the recirculation tank, it is still proposed that nitrification process occurred in the recirculation tank. These results are consistent with those of Mosely (2001) who also observed NH_4^+-N removal in the recirculation tank that was concluded to have occurred through the nitrification process in the recirculation tank. As well, Mosley (2001) found that dissolved oxygen of 2 to 5 mg/ L could be present in the recirculation tank under which nitrification process could occur.

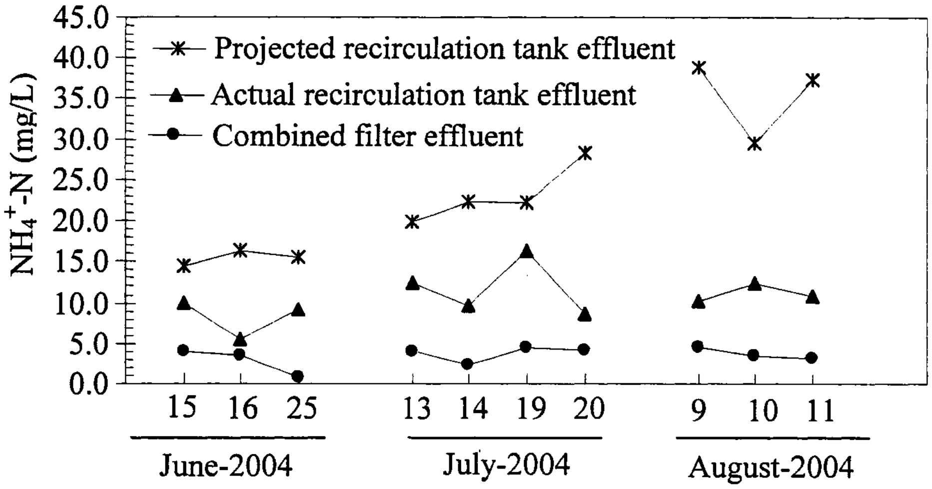


Figure 6.13 Comparison of projected and actual NH_4^+-N concentrations in the recirculation tank and measured filter effluent NH_4^+-N concentrations

The collective results presented in Figure 6.13 also suggest that recirculation tank has two important functions: (i) a bioreactor for contaminant removal as described in Chapter 5 and (ii) a mixing chamber to provide stable nutrient loading rates to the filter bed due to the biological reaction and dilution from filter effluent. The septic tank effluent NH_4^+-N increased over time from June to July, 2004 as shown in Appendix F-Septic Tank Effluent. However, Figure 6.13 shows that the actual measured NH_4^+-N was quite stable although the projected NH_4^+-N in the recirculation tank and the septic tank effluent NH_4^+-N were highly variable. Consequently, the filter beds could produce consistent effluent NH_4^+-N during the whole research courses as shown in Figure 6.13.

BOD_5 removal

As described in the Chapter 5, it was hypothesized that the aerobic removal of BOD_5 can occur in the recirculation tank due to the increase in oxygen concentrations resulted from the recycling of filter effluents. Figure 6.14 shows the projected and actual measured BOD_5 in the recirculation tank and the filter effluent BOD_5. The projected BOD_5 in the recirculation tank was calculated using Equation 5.1 and the filter effluent BOD_5 was measured from samples taken from the combined filter bed effluents of the four field-scale RBFs.

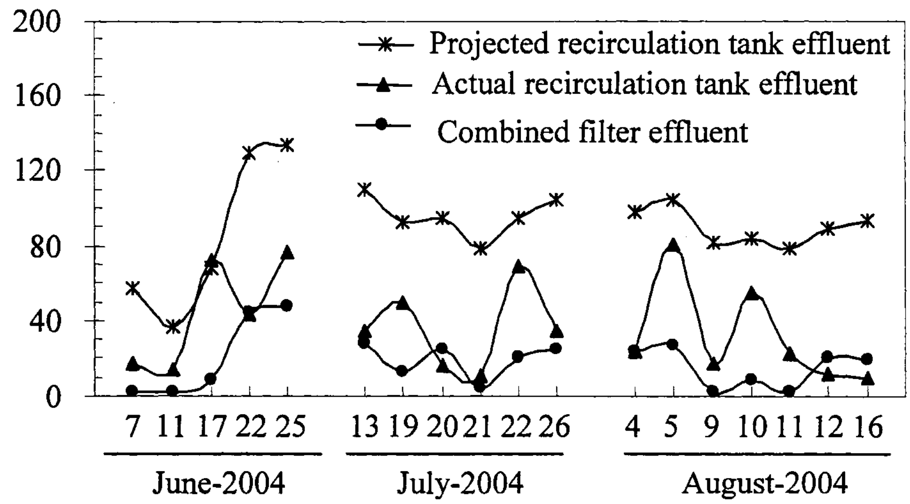


Figure 6.14 Comparison of projected and actual BOD_5 concentrations in the recirculation tank and measured BOD_5 concentrations in the combined filter effluent

Figure 6.14 shows that there was a difference between the projected and the actual measured BOD_5 concentrations in the recirculation tank. The average projected recirculation tank BOD_5 concentration was calculated to be 90 ± 23 mg/ L and the actual measured BOD_5 concentration in the recirculation tank was found to be 36 ± 25 mg/ L. Based these data, approximately 60% BOD_5 reduction was achieved in the recirculation tank. The occurrence of BOD_5 removal in the recirculation tank in the field-scale study is consistent with the conclusion previously described in Chapter 5 and the results from Mosley (2001). Figure 6.14 also shows that the actual measured recirculation tank effluent BOD_5 was more variable as compared with projected concentrations. However, the combined filter effluent BOD_5 concentration ranged between 18 ± 14 mg/ L, demonstrating the robustness of the filters to handle variable BOD_5 loadings. The average filter effluent BOD_5 was 18 ± 14 mg/ L. Based on these data, approximately 50% of the BOD5 removal was achieve within the filter beds as compared to the recirculation tank effluent (36 ± 25 mg/ L).

The collective results of the experiments support the theory that the recirculation tank can also function as a bioreactor to reduce organic loading rates to the filter bed. In particularly, the organic loading was decreased from 0.060 kg/ m^2-day in the septic tank effluent stream to 0.006 kg/m^2-day in the actual measured recirculation tank effluent stream, indicating that organic loading to the filter bed was reduced by a factor of 10 due to the reaction, dilution, and mixing in the recirculation tank. Organic loading rates are also applied for intermittent packed bed filters (including RBFs and single pass packed bed filters) design. Since organic loading rates are determined by multiplying organic concentration (BOD_5) in the septic tank effluent and hydraulic loading rate (HLR) with appropriate unit conversion, RBFs can be dosed with higher HLRs than single pass packed bed filter due to the decrease of organic loadings in the recirculation tank. These data and analysis confirm the advantage of multiple-pass packed bed filters, or RBFs over single-pass packed bed filters. In particular, RBFs would require less footprint than single-pass packed bed filters due to the fact that footprints are determined by wastewater flowrate divided by HLRs.

6.4.4 Mass Balance Analysis in the Recirculation tank

Overview of Mass Balance Analysis

Mass balance is key to the design and analysis of microbiological processes. As presented in Figure 6.15, a definition of the system boundary around the recirculation tank (as represented by the dashed line) allows for a mass balance to be conducted around this part of the system.

The establishment of this system boundary defines the input streams (septic tank effluent and recycled filter effluent) and output stream (recirculation tank effluent). The definition of the system in this way allows for mass balances on flow rates or constituents in the flow streams (i.e, water quality parameters) to be conducted. The consumption or generation of mass within the system boundaries are also taken into account to reflect chemical or biological reactions that may be occurring within the system boundaries. For this research, mass balances around the recirculation tank were conducted to provide a more comprehensive evaluation of the parameters measured during the field-scale RBF study.

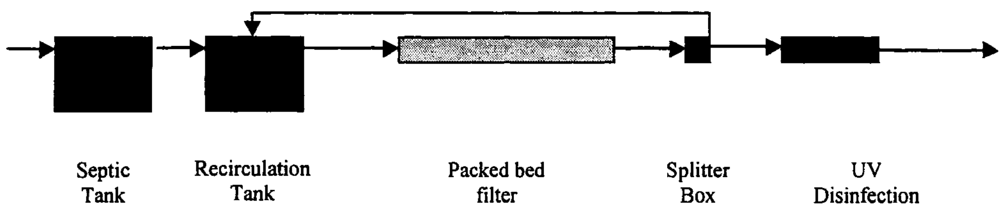


Figure 6.15 Mass balance around recirculation tank in RBF system

Once a reactor system, system boundary and the components on which to do the mass balances are selected, a general mass balance equation can be written (Equation 6.1).

$$\text{Accumulation} = \text{Input} - \text{Output} + \text{Generation} - \text{Consumption} \qquad [6.1]$$

According to Rittmann and McCarty (2001), accumulation is the total mass of the component in the system, or the product of the volume times the concentration. The rate term takes the general mathematical form of d(VC)/ dt, in which V is the volume of the control volume, C is the components concentration, and d /dt is the differential with respect to time.

Bioreactor Definition of the Recirculation Tank

The basic reactor models used in environmental applications include the batch reactor, the continuous stirred tank reactor and plug-flow reactor as shown in Figure 6.16.

The simplest suspended growth reactor is the batch reactor. In this system, the reactor is filled with appropriate proportions of the liquid or slurry stream to be treated, the bacterial cultural to be used and the required nutrients, such as nitrogen and phosphorus. The reactor contents are then stirred if needed to keep the reactor contents in suspension. Typically, batch reactor designs are used to achieve high removal efficiencies of individual wastewater constituents (Rittmann and McCarty, 2001).

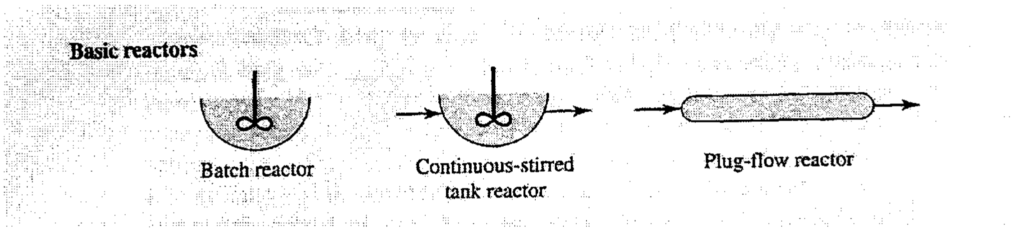


Figure 6.16 Basic reactor models in environmental applications (Source: Rittmann and McCarty, 2001)

The second basic reactor type is the continuous-flow stirred-tank reactor (CSTR) which is also commonly called a completely mixed reactor. In this system, the liquid or slurry stream is continuously introduced into the reactor volume while the liquid contents are continuously removed from the reactor. The basic characteristic of the ideal CSTR is that the concentrations of substrates and microorganisms are the same throughout the reactor volume. In addition, the substrate and microorganism concentrations in the effluent stream are the same as the concentrations contained in the reactor volume (Rittmann and McCarty, 2001).

The third basic reactor type is the plug-flow reactor (PFR). In the ideal PFR, the influent flow moves through the reactor with no mixing with earlier or later entering flows. Hence, an element of the stream entering at one time moves down the reactor as a *'plug'* in a discrete manner. An ideal PFR is difficult to realize in practice because mixing in the direction of flow is impossible to prevent (Rittmann and McCarty, 2001).

The basic reactors noted above are frequently combined in series or in parallel in full-scale treatment design. Reactors in series are often used when different types of treatment are needed, such as organic oxidation followed by nitrification. Designs that place reactors in parallel are used at most treatment plants to provide redundancy in the system. This type of design compensates for downtime that may occur when reactors are out of service.

A conceptual model of the RBF system evaluated in this study is presented in Figure 6.17. For purposes of mass balance analysis, a system boundary is defined around the recirculation tank. During the dosing operational periods, the septic tank effluent and the recycled filter effluent streams are continuously pumped into the recirculation tank while the recirculation tank effluent stream is continuously pumped into the filter. As such, during the dosing period, the recirculation tank can be modeled as a CSTR reactor (Figure 6.17A).

During the operational time between two doses, all of the dosing pumps are off. Therefore, there is no input or output into the recirculation tank system. As such, during this operating time, the recirculation tank can be modeled as a batch reactor (Figure 6.17.B).

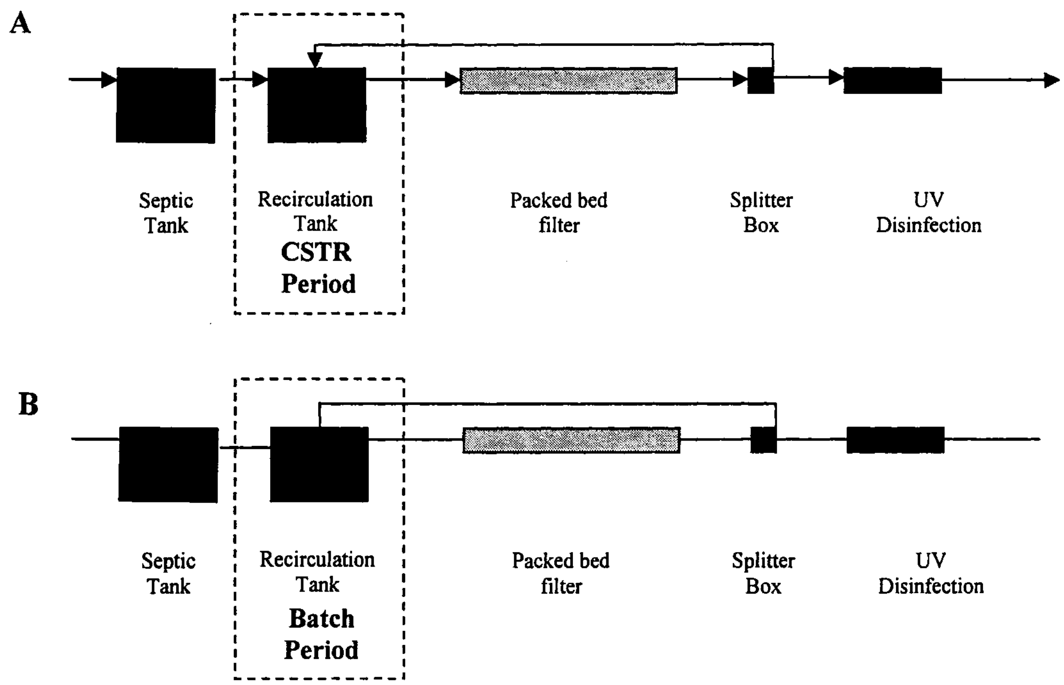
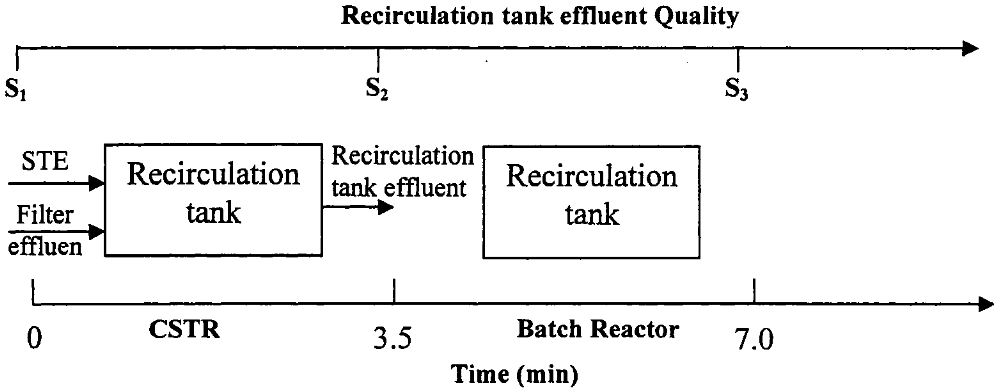


Figure 6.17 Reactor design for the recirculation tank: A) dosing operation; and B) between doses.

Mass Rate of Substrate Accumulation in the Recirculation Tank



S_1: Projected recirculation tank water quality;
S_2: Recirculation tank water quality at the end of a dose;
S_3: recirculation tank water quality at the beginning of a dose

Figure 6.18 Bioreactor arrangements for the recirculation tank over time

Based on the operational period shown in Figure 6.18 and Equation 6.1, two mass balances that describe the recirculation tank system can be defined (Equation 6.2):

<u>Dosing Period – CSTR Model</u>

$$V\frac{dS_r}{dt} = Q_s S_s + Q_f S_f - Q_r S_r + \gamma_{ut,1} V \qquad [6.2]$$

where,

V:	volume of the recirculation tank, L
S_s S_f, & S_r:	component concentrations in the septic tank effluent, filter effluent, and the recirculation tank effluent streams, mg/ L
t:	time step, day;
Q_s, Q_f & Q_r:	flow rates of septic tank effluent, filter effluent, and the recirculation tank effluent streams, L/ day
$\gamma_{ut,1}$:	rate of substrate utilization in the CSTR period, mg/ L-day.

As previously described in Section 7.3.1, the dosing volume of one orifice is 5.7 L/ dose; and there are 150 orifices. Therefore, the projected total volume of combined water stream (septic tank effluent and recycled filtration effluents) is 4275 L/ dose (5.7 L/ dose/ orifice × 150 orifice × (4+1) = 4275 L /dose). Section 7.3.1 of this chapter also introduced that the total volume of the recirculation tank was 11,350 L. The recirculation tank is designed to either remain full or be pumped down during periods of low wastewater flows. Since doses to the recirculating filter are of a constant volume and occur at timed intervals, the water level in the recirculation tank will rise and fall in response to septic tank effluent flow, return filtrate flow, and filter dosing. In order to simplify the variation of water level in the recirculation tank, this chapter assumes that the recirculation tank always remains full volume of water. Therefore, the volume of water entering into the recirculation tank every dose was about 38% of the recirculation tank volume. In another words, the water quality of combined influent into the recirculation tank every dose is not expected to significantly change the water quality in the recirculation tank by dilution only. Due to the physical restriction of recording water quality in the recirculation tank at the beginning and at the end of a dosing period, this chapter assumes that the CSTR reactor can achieve a steady-state at which time the change of mass accumulation becomes zero, or V dS/ dt = 0. In addition, S_r in Equation 6.2 can be replaced by S_2 as shown in Figure 6.18. Therefore, Equation 6.2 can be rewritten as:

$$0 = Q_s S_s + Q_f S_f - Q_r S_2 + \gamma_{ut,1} V \qquad [6.3]$$

<u>*Non-Dosing Period: Batch Reactor Model*</u>

$$V \frac{dS_r}{dt} = V \gamma_{ut,2} \qquad [6.4]$$

where

$\gamma_{ut,2}$ is the rate of substrate utilization in batch reactor period, mg/L-day

Based on the time frame in Figure 6.18 and the recirculation tank effluent water quality, Equation 6.4 can be rewritten as:

$$\frac{S_3 - S_2}{\Delta t} = \gamma_{ut,2} \qquad [6.5]$$

Both Equation 6.3 and 6.5 include S_2, which is the recirculation tank effluent quality at the end of the dosing period. As well, due to the short dosing cycle periods, the CSTR model was assumed to have negligible biological and chemical reactions occurring. Therefore, the recirculation tank was assumed to function as a dilution and mixing chamber for the septic tank effluent and recycled filter effluent streams into the recirculation tank during the dosing periods. Based on this assumption, $\gamma_{ut,1}$ in Equation 6.3 can be set as zero and S_2 can be defined as presented in Equation 6.6.

$$S_2 = \frac{Q_s S_s + Q_f S_f}{Q_r} \qquad [6.6]$$

Utilization Rate (γ_{ut}). Commonly, the rate of substrate utilization is assumed to following Monod kinetics, as given by Equation 6.7 (Rittmann and McCarty, 2001).

$$\frac{dS_r}{dt} = -\frac{\hat{q} X S_r}{K_s + S_r} \qquad [6.7]$$

where,

X: concentration of active biomass, mg/ L

$\hat{q}$: the maximum specific rate of substrate utilization, day^{-1}

K_s: concentration giving one-half the maximum rate, mg/ L.

Due to the lack of data of active biomass concentrations, or X as shown Equation 6.7, this chapter assumes that the utilization rate, γ_{ut}, can be modeled as a first order reaction with component concentration of interest in the recirculation tank, as given by Equation 6.8 (Crites and Tchobanoglous, 1998).

$$\gamma_{ut} = -kS_r \qquad [6.8]$$

where,

k: reaction rate coefficient, day^{-1}.

The first-order reaction rate for BOD_5 and nitrogen removal has been applied in onsite wastewater treatment systems, including lagoon systems (Crites and Tchobanoglous, 1998) and wetland systems (Crites and Tchobanoglous, 1998; Kadlec and Knight, 1995). The reaction rate of components of interest can be determined by the combination of Equation 6.5, 6.6 and 6.8, as given by Equation 6.9.

$$S_3 = S_2 e^{-kt} \tag{6.9}$$

Data Input. Table 6.2 summaries the input data used in the mass balance analysis. In this chapter, three different wastewater parameters were evaluated, including TN, NH_4^+-N, and BOD_5. Based on the average daily flow rate of 10,000 L/ day (10 households with 1,000 L/ day for each household) and the recirculation ratio of 4:1, the flow rate of filter effluent (Q_f) and the flow rate of recirculation tank effluent (Q_r) were calculated to be 40,000 and 50,000 L/ day, respectively. To minimize the error for reaction rate coefficient determination, average concentrations of components of the different wastewater streams (septic tank effluent, filter effluent, and the recirculation tank effluent) over the whole experiment course were applied.

Table 6.2 Summary of data input for mass-balance analysis

Parameters		Values
Q_s		10,000 L/ day
Q_f		40,000 L/ day
Q_r		50,000 L/ day
TN	S_s	149.1 mg/ L
	S_f	36.0 mg/ L
	S_3	36.8 mg/ L
NH_4^+-N	S_s	103.8 mg/ L
	S_f	3.4 mg/ L
	S_3	16.3 mg/ L
BOD_5	S_s	381 mg/ L
	S_f	17 mg/ L
	S_3	36 mg/ L

Reaction Rate of Components of Interest. The reaction rate coefficients for TN, NH_4^+-N and BOD_5 were calculated using the data presented in Table 6.2 and Equations 6.9 as shown in Table 6.3.

Table 6.3 Summary of components reaction rates in the recirculation tank

Components of interest	k, day^{-1}
TN	0.41
NH_4^+-N	0.32
BOD_5	0.80

The average decay coefficient of BOD_5 for the activated-sludge process for domestic wastewater treatment is 4 to10 day^{-1}, with a typical value of 4 day^{-1} (Crites and Tchobanoglous, 1998). In the present study, the decay coefficient for BOD_5 was found to be 0.8 day^{-1} as shown in Table 6.3, which is much smaller than the value for activated sludge process. However, the typical reaction rate coefficient of BOD_5 for the lagoon system is approximately 0.1 to 0.3 day^{-1} (Crites and Tchobanoglous, 1998), which is lower than the coefficient value of the present study. The average decay coefficient of NH_4^+-N is within reported decay coefficients, which are between 0.20 to 0.90 day^{-1} (MetCalf & Eddy, 2003). However, more researches need to be conducted to investigate the mass balance in the recirculation tank and the decay coefficient of components of interest.

6.5 UV Reactor

Disinfection is considered to be the primary mechanism for the inactivation/destruction of pathogenic organisms to prevent the spread of waterborne diseases. An Ultraviolet (UV) disinfection system transfers electromagnetic energy from a mercury arc lamp to an organism's genetic material (DNA and RNA). When UV radiation penetrates the cell wall of an organism, it destroys the cell's ability to reproduce. UV radiation, generated by an electrical discharge through mercury vapor, penetrates the genetic material of microorganisms and retards their ability to reproduce.

The effectiveness of a UV disinfection system depends on the characteristics of the wastewater, the intensity of UV radiation, the amount of time the microorganisms are exposed to the radiation, and the reactor configuration. For any one treatment plant, disinfection success is directly related to the concentration of colloidal and particulate constituents in the wastewater (USEPA, 1999). The most useful variable that can be readily controlled and monitored is Total Suspended Solids. TSS has a direct impact on UV disinfection, which is related to the level of pretreatment provided (USEPA, 2002).

The effectiveness of UV disinfection is dependent upon UV power, contact time, liquid film thickness, wastewater absorbance, wastewater turbidity, system configuration, and temperature. Empirical relationships are used to relate UV power (intensity at the organism boundary) and contact time. Since effective disinfection is dependent on wastewater quality as measured by turbidity, it is important that pretreatment provide a high degree of suspended and colloidal solids removal.

Although high log reductions of fecal coliform could be achieved by the recirculating biofilters in this study, they did not meet the local regulated levels of less than 200 CFU/ 100mL. Therefore, an UV reactor chamber was connected to inactivate the pathogenic microorganisms in the filtration effluents as shown in Figure 6.1, and the UV reactor was designed to provide 99.9% reduction in bacteria and a bacterial fecal colony count of less than 200 per 100 mL.

However, it was observed that the UV reactor did not provide sufficient removal to meet the local regulation in this field study. In stead, the UV reactor in the present study reduced the fecal coliform colony from the combined filter effluents of 1.12×10^6 to 3.87×10^6 colony count per 100 mL in the UV reactor effluent, indicating that the UV reactor in this study provided 1.5 log-reductions for fecal coliform removal. UV has been successfully applied by Babcock et al. (2004) to disinfect the aerobic reactor effluent. In their study, the effluent was passed through a UV unit ten times, achieving 2 log reduction every time to produce a final effluent containing less than 2 CFU/100mL. Therefore, UV treatment unit optimization would be a possible approach to achieve stringent fecal coliform inactivation standards.

6.6 Summary and Conclusion

This chapter evaluated the long term effectiveness of four types of filter media (silica sand, crushed glass, peat, and geotextile) in RBFs for the treatment of wastewater from multiple residences. Organic carbon removal, nutrient removal, TSS removal, and pathogenic microorganism inactivation were the measured parameters. The results showed that crushed glass could be applied as an alternative medium to silica sand for RBFs, since the crushed glass filter could achieve highly stable BOD_5, TSS, and NH_4^+-N removal. Geotexitle was found to be another option for an effective RBF medium, even though the effluent BOD_5, TSS, and NH_4^+-N levels were slightly higher than those of crushed glass filter effluent. This chapter found that peat was not an effective RBF medium due to the poor BOD_5 and NH_4^+-N removal. In addition, this chapter also found that the RBFs were not able to remove TP and fecal coliform effectively.

This chapter investigated the performance of RBFs by evaluating the individual components, including the recirculation tank, filter bed, and the UV reactor. It was found that the recirculation tank was the main component in which TN removal took place. This conclusion indicated that further optimization of the recirculation tank could improve TN removal. Meanwhile, aerobic BOD_5 and nitrification processes were also found to take place in the recirculation tank. These results indicate that the recirculation tank functioned as a bioreactor chamber for biodegradation, dilution, and mixing of water quality.

In addition, this chapter conducted mass balance analysis for TN, NH_4^+-N, and BOD_5 in the recirculation tank. The reaction rate coefficients of these three components of interest in the recirculation tank were provided in this chapter. Finally, this chapter found that the UV unit was inefficient for fecal coliform inactivation to achieve local regulation of effluent fecal coliform less than 200 CFU per 100mL, indicating that more investigations should be conducted to optimize the disinfection process for RBFs.

7. A NOVEL DUAL-MEDIA DESIGN FOR RBFs

7.1 Abstract

The main objectives of this study were to: (1) examine the efficiency of a new media design (dual media) recirculating biofilters (RBF) for domestic wastewater treatment with crushed glass and geotextile and (2) evaluate the performance of this novel dual media RBF. The parameters discussed in this study include the volume ratios between crushed glass and geotextile and hydraulic loading rate (HLR).

Ten bench scale filters were operated containing geotextile and crushed glass at different volume ratios, including geotextile volume percentages of 0 %, 25 %, 50 %, 75 % and 100 %. Each of these five RBFs was dosed at two HLRs, including 0.20 and 0.40 $m^3/m^2/d$. Seven water quality parameters were monitored, including BOD_5, NH_4^+-N, NO_3-N, TN, turbidity, pH, and fecal coliform. The results show that the dual media RBF with geotextile volume percentage of 25 % produced the highest removals for BOD_5, turbidity, and fecal coliform of 93.6 %, 94.8 % and 3.4-log reduction, respectively. This chapter also quantified the BOD_5 removals within the filter beds and the recirculation tanks for all tested RBFs. The results showed that the dual media RBF with the geotexitle volume percentage of 25 % provided superior BOD_5 removal within the filter bed as compared to the geotextile RBF. In addition, this dual media RBF also provided higher BOD_5 removal in the recirculation tank than the crushed glass RBF. Since the total BOD_5 removal by a complete RBF system includes the BOD_5 removals from both the filter bed and the recirculation tank, the dual media RBF with the geotextile volume percentage of 25 % could provide the most efficient total BOD_5 removal amongst all tested RBFs in this chapter.

7.2 Introduction

With the increasing housing development stresses, limited land resources has become a concern for traditional onsite wastewater treatment systems. USEPA (2002) showed detailed system design guidelines for RBFs, including hydraulic loading rates (HLRs) for different filter media, and recommended size specifications for septic and recirculating tanks. The design guideline indicates that the system footprint such as filter surface area can be decreased by increasing hydraulic loading rates (HLRs) for a given amount of daily flow of domestic wastewater.

Chapter 4 and 6 found that recirculating geotextile biofilters can provide efficient performance for domestic wastewater treatment based on both bench and field-scale studies. In other studies, textile filter has been shown to present an advantageous design in terms of increased HLR capacity as compared to recirculating sand filters. Loomis et al. (2001) found that a recirculating textile filter (RTFs) dosed with 0.76 $m^3/m^2/day$ of domestic wastewater could provide removals for BOD_5 and TSS of 99 % and 80 %, respectively. Roy and Reid (1998) also reported that HLRs for textile RBFs could be as high as 1.00 to 1.80 $m^3/m^2/day$. They found that the surface area of textile filter for treating 100,000 L/ day of domestic wastewater was only 55 to100 m^2, however the surface areas required for conventional leach field and RSFs for treating same volume of domestic wastewater were 2,500 and 500 m^2, respectively. Therefore, the characteristics of textile provide an interesting research gap in that textile RBFs have the potential to decrease the land area demand significantly by being operated under much higher HLRs than RSFs.

Although RTFs are efficient for onsite wastewater treatment, they can become relatively costly or difficult to implement since textile is subject to local availability and transportation sensitivity (Wren et al., 2004; USEPA, 2002). Therefore, it is of great practical interest to develop a modified RTF which is less expensive and more efficient. Dual-media filters provide an opportunity to maintain the high performance while minimizing cost by introducing another less expensive and more readily available medium. In dual media filtration, the fine media is placed under the coarse media, to allow for better particle penetration across the filter and lower headloss (Droste, 1997). Thus the fine media in the bottom layer is able to provide good performance with less possibility of clogging. Compact filters are an on-site wastewater system used extensively in France for the past 12 years (Joy et. al. 2004). These filters contain two layers, with coarse particles at a depth of 2 to5 mm on top of fine particles at a depth of 0.5 to 2 mm. Joy et al. (2004) reported that in RSFs treatment occurs only in the upper portion of the filter bed whereas in a compact filter, treatment occurs over the entire depth of the media.

Crushed glass has been previously addressed as an alternative medium to sand for RBFs (Darby et al., 1996; Emerick et al., 1997). Crushed glass has been used successfully as a RBF medium to treat septic tank effluent (Elliott, 2001). The same reference also reported that crushed glass was produced from pulverizing recycled glass and this environmentally friendly medium is approximately \$12.50/ ton less expensive than traditional or natural sand. Thus, it is expected that dual-media RBFs consisting of textile and crushed glass could be significantly less expensive than the RTFs due to the low cost of crushed glass.

Therefore, the main objectives of this chapter were to: (i) examine the efficiency of a new dual media RBFs for domestic wastewater treatment with crushed glass and geotextile fiber; and (ii) investigate and evaluate the performance of all tested novel dual media RBFs based on seven water quality parameters, including BOD_5, NH_4^+-N, NO_3^--N, TN, TSS, pH, and fecal coliform.

7.3 Materials and Methods

7.3.1 Description of Dual-Media Bench-Scale RBFs

The setup of bench-scale RBFs has previously been described in detail in Chapter 3. The main setup of the bench-scale dual-media RBFs was similar to the sketch in Chapter 3 with the exception of the composition of the filter media. Six dual-media and four single media bench-scale RBFs were conducted based on the setup as shown in Figure 7.1. The ten 15 cm filter columns were filled with crushed glass and geotextile with different volume ratios, including 100 % crushed glass, 25 % geotextile and 75 % crushed glass, 50 % geotextile and 50 % crushed glass, 75 % geotextile and 25 % crushed glass, and 100 % geotextile. Two HLRs were applied for all these RBFs, including 0.20 and 0.40 m^3/m^2 per day. The recycle rate of 4:1 and dose frequency of 96 times per day were applied to all of the bench-scale RBFs. Similar as the dosing on/off intervals described in the Chapter 3, each dose lasted 4 min and the interval time between two doses was 11 min.

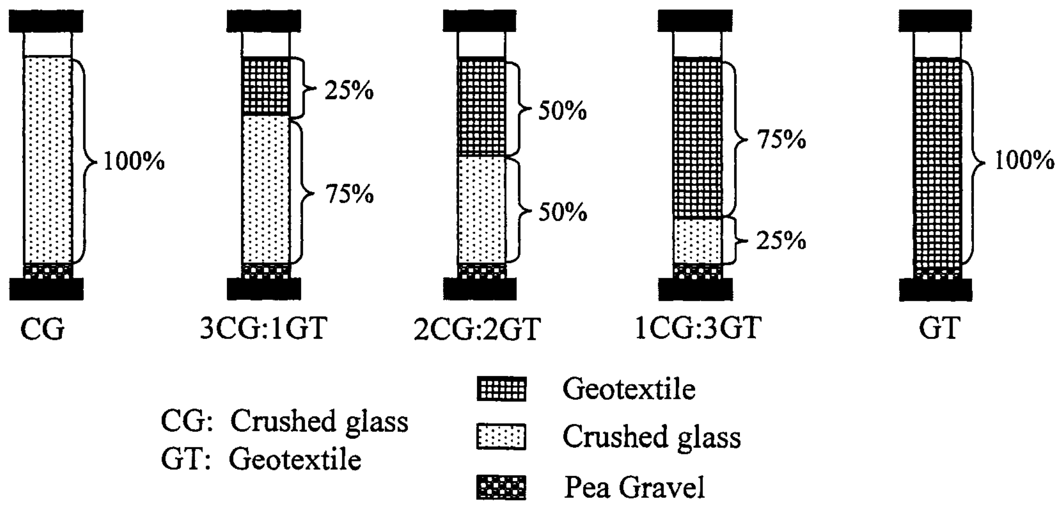


Figure 7.1 Dual-media bench-scale RBFs.

7.3.2 Sampling and Water Quality Parameters Measurements

Samples were collected with the sampling frequency described in Chapter 3 from the septic tank, the recirculation tank, and the filter effluents; and were analyzed for BOD_5, TN, NH_4^+-N, NO_3-N, pH, fecal coliform, and turbidity were determined using methods described in detail in Chapter 3. Seven water quality parameters were measured for septic tank effluent, including BOD_5 of 160 ± 37 mg/ L, turbidity of 41.4 ± 15.2 mg/ L, TN of 39.8 ± 5.5 mg/ L, NH_4^+-N of 36.7 ± 5.4 mg/ L, NO_3^-N of 1.4 ± 0.8 mg/ L, fecal coliform of $6.7\times10^5 \pm 3.6\times10^5$/100 mL, and pH of 7.7 ± 0.1.

7.3.3 Biofilm Profile Analysis

The surface structure of biofilm around the filter media in the RBFs was examined by scanning electronic microscope (SEM) (Hitachi 2700). Samples were taken at three different depths for both crushed glass and geotextile layers, including top, middle, and bottom. The crushed glass and geotextile were sampled and dried at room temperature for 24 hours before SEM images were taken.

7.4 Results and Discussion

Figure 7.2 shows the effluent BOD_5 concentrations of the bench-scale RBF with media distribution of 3CG:1GT (or 75 % crushed glass and 25 % geotextile) dosed at the high HLR (i.e., 0.40 $m^3/m^2/d$). The results demonstrate that this bench-scale RBF achieved a pseudo-steady-state condition after two weeks of operation, with effluent BOD_5 concentrations leveling out to less than 20 mg/ L after Day 17 of the trials. This effluent BOD_5 concentration was obtained even through the septic tank effluent BOD_5 concentration increased from 120 to 240 mg/ L. The other bench-scale RBFs demonstrated the same trend as presented in Figure 7.2; and are presented in Appendix G – Biofiltration Effluent BOD_5. The discussion presented herein was based on the measured results during pseudo-steady-state period (i.e., after and including Day 17). In particular, Appendix G reported all measured water quality during the pseudo-steady-state period; and the data shown in Appendix G were all employed for systems performance evaluation and comparison.

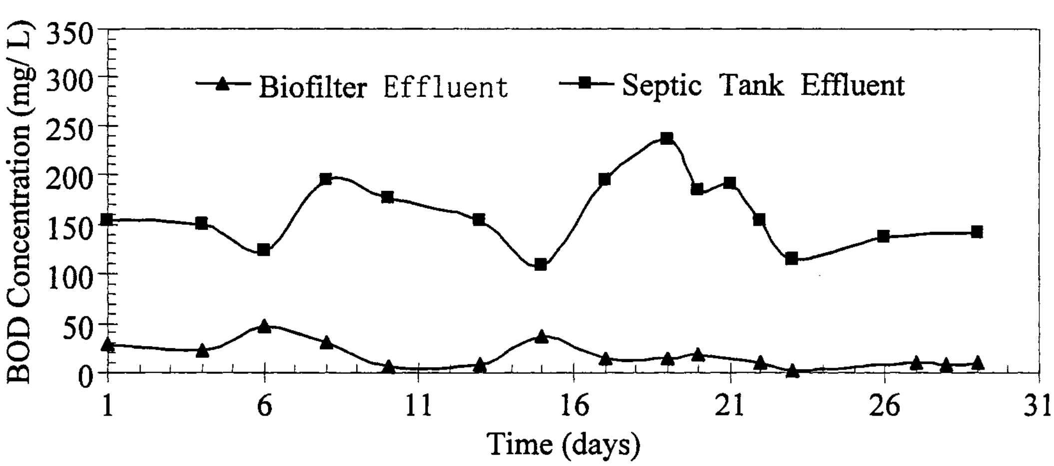


Figure 7. 2 Dual-media RBF of 3CG:1GT.

7.4.1 Contaminants Removals in Bench-Scale RBFs

Contaminant removals in the ten pilot-scale RBFs based on BOD_5, NH_4^+-N, Turbidity, and fecal coliform removals.

Turbidity Removal

Figure 7.3 shows that the filters containing greater than 75 % of geotextile resulted in a high effluent turbidity at high HLRs (i.e., 0.40 m^3/m^2/day). This finding indicates that turbidity removal occurred primarily in the crushed glass layer at high HLRs. For low HLRs (i.e., 0.20 m^3/m^2/day) effluent turbidity concentrations were higher in the filters containing 100 % geotextile media than for the filters containing less than 50 % geotextile. Thus the fine crushed glass media was critical in providing improved turbidity removal, although it was not required to fill the entire column. Consequently having approximately half of the column filled with geotextile would not compromise turbidity removal.

Single media RBFs. The crushed glass RBFs produced average effluent turbidity concentrations of 1.5 ± 0.1 mg/ L and 2.6 ± 0.6 mg/ L at low and high HLRs, respectively as shown in Figure 7.3. As presented in Figure 7.3, average effluent TSS concentrations were 1.1 ± 0.3 mg/ L and 12.1 ± 8.2 mg/ L at low and high HLRs, respectively. An ANOVA analysis showed that there was a significant difference amongst these four single media RBFs in terms of turbidity removal (α=0.05), as shown in Appendix: J – ANOVA – Single Media – Turbidity.

Dual media RBFs. The average effluent TSS concentration varied in the six dual media RBFs (Figure 7.3). The effluent TSS at low and high HLRs were 1.3 ± 0.3 and 1.1 ± 0.3 mg/ L, 1.8 ± 0.7 and 9.3 ± 3.2 mg/ L, and 3.5 ± 2.9 and 4.1 ± 2.4 mg/ L for dual media RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively. An ANOVA results showed that there was a significant difference between the six dual media RBFs for TSS removals at the confidence level of 95%, as shown in Appendix: J – ANOVA – Dual Media – Turbidity.

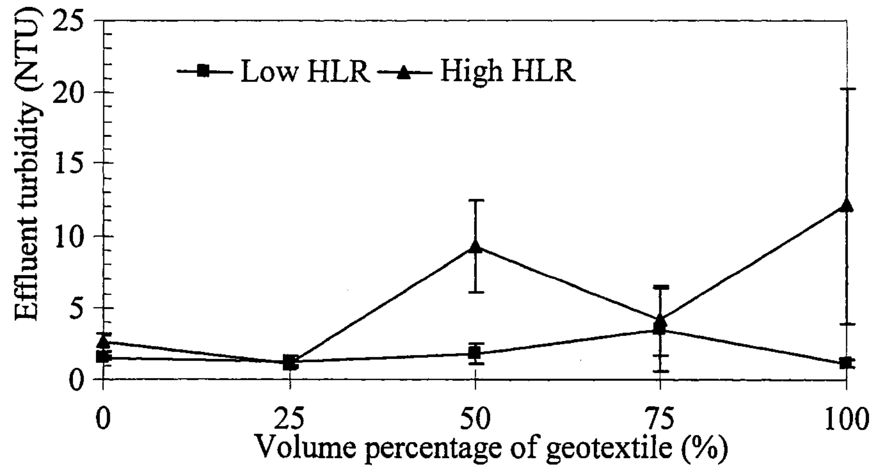


Figure 7. 3 Effluent turbidity concentration with various ratios of geotextile as filter media (*Error bars indicate standard deviation from mean*)

BOD₅ Removal

Figure 7.4.A and B show the BOD_5 concentrations in the bench-scale RBF effluents and the recirculation tank effluents for all tested RBFs at HLRs of 0.20 and 0.40 $m^3/m^2/d$. In particular, the crushed glass and the geotextile RBFs shown in Figure 7.4 were the bench-scale RBFs with the geotextile volume percentages of 0 % and 100 %, respectively. The dual media RBFs shown in Figure 7.4 were the bench-scale RBFs with the geotextile volume percentages of 25, 50 and 75 %.

Single media RBFs. The average effluent BOD_5 concentrations measured for the two crushed glass RBFs were 12 ± 6 mg/ L and 16 ± 6 mg/ L at the low and high HLRs, respectively (Figure 7.4). The two geotextile RBFs produced average effluent BOD_5

concentrations of 11 ± 4 mg/ L and 24 ± 2 mg/ L at the low and high HLRs, respectively. The results of an ANOVA analysis showed that there was a significant difference amongst these four single media RBFs in terms of BOD_5 removal (α=0.05), as shown in Appendix J – ANOVA – Single Media – BOD_5.

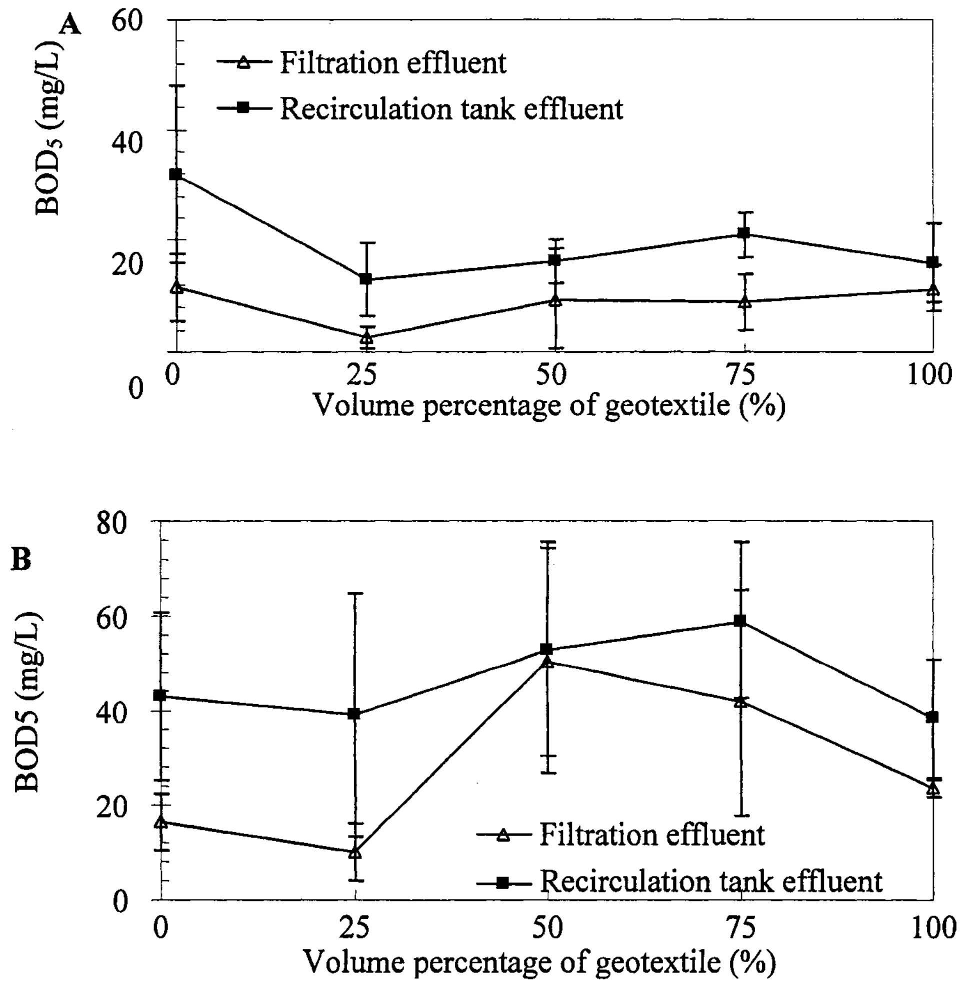


Figure 7.4 Effluent BOD_5 concentrations in the recirculation tank and RBF effluents at A) low HLR and B) high HLR. (Error bars indicate standard deviation from mean).

Dual media RBFs. For RBFs operated at a low HLR, Figure 7.4.A shows that the RBF effluent BOD_5 concentrations averaged 3 ± 2, 10 ± 9, 9 ± 5 mg/ L for the dual media

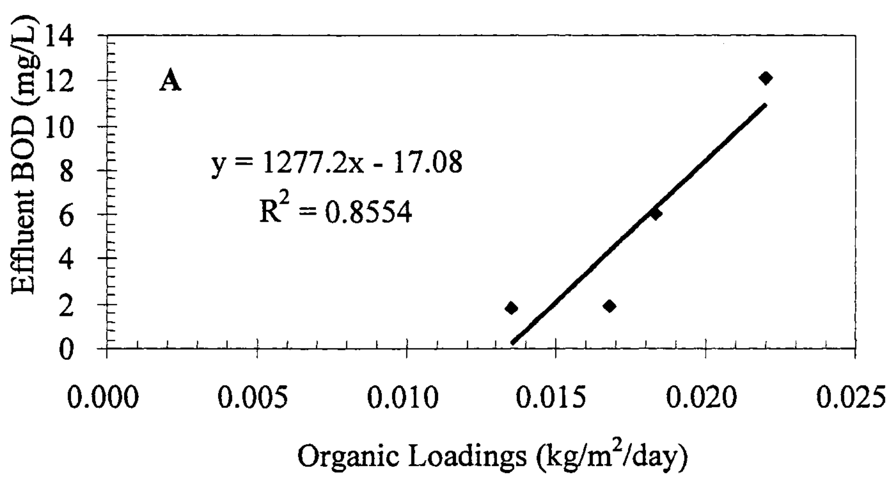
RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively. For RBFs operated at a high HLR, Figure 7.5.B shows that the RBF effluent BOD_5 concentrations averaged were 10 ± 6, 51 ± 24, 42 ± 24 mg/ L for the dual media RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively. These results demonstrate that the average RBF effluent BOD_5 concentrations were highly variable amongst the different dual media RBFs tested in this study. ANOVA results showed that there was a significant difference between the six dual media RBFs for BOD_5 removal (α=0.05), as shown in Appendix: J – ANOVA – Dual Media – BOD_5. More importantly, Figure 7.4 shows that the dual media RBF with the geotextile volume percentage of 25 % produced an average effluent BOD_5 concentration less than the 100 % crushed glass RBF.

Figure 7.4.A shows that the RBF effluents BOD_5 concentrations were generally lower than the recirculation tank effluent BOD_5 concentrations for all of the experimental RBFs dosed at a low HLR. Figure 7.4.B also shows a similar observation for all of the bench-scale RBFs dosed at a high HLR. These findings indicate that BOD_5 removal occurred within the filter beds of all the RBFs tested in this chapter. For the single media RBFs, Figure 7.4 shows that BOD_5 removal within the filter bed of the crushed glass RBF was higher than within the geotextile RBF at both low and high HLRs. This finding is possibly related to the high hydraulic retention time in the crushed glass filter bed due to the lower porosity of the crushed glass than that of the geotextile. For the dual media RBFs, it was also observed that the BOD_5 removal within the filter bed was higher under the operation of a low HLR than a high HLR. Again, a possible reason for this observation is that a low HLR provides a longer hydraulic retention time than operating at a high HLR. USEPA (2002) proposed that BOD_5 could be nearly completely removed if the wastewater retention time in the sand media is sufficiently long for the microorganisms to degrade waste constituents.

Organic Loading Rates. Since RBFs are predominately aerobic biological treatment units, it is appropriate that they are designed based on organic loading rates (USEPA, 2002). USEPA (2002) recommends that BOD_5 loading rates in sand media

RBFs should not exceed approximately 0.024 kg/m^2/day when the effective size of the sand media is approximately 1.0-mm and the dosing rate is at least 12 times per day. Crites and Tchobanoglous (1998) suggested that typical organic loading rates range from 0.01 to 0.04 kg/m^2/day for recirculating granular medium filters.

As described in Chapter 3, Equation 3.7 shows that calculation method of organic loading (BOD_5) rates applied in this study. During the period of bench-scale dual media RBFs test, variable septic tank effluent BOD_5 concentrations were observed. Therefore, the dual media RBFs were dosed with variable organic loading (BOD_5) rates, since the constant HLRs were maintained. Therefore, this study compared the organic loading rates from the septic tank effluent and the biofiltration effluents.



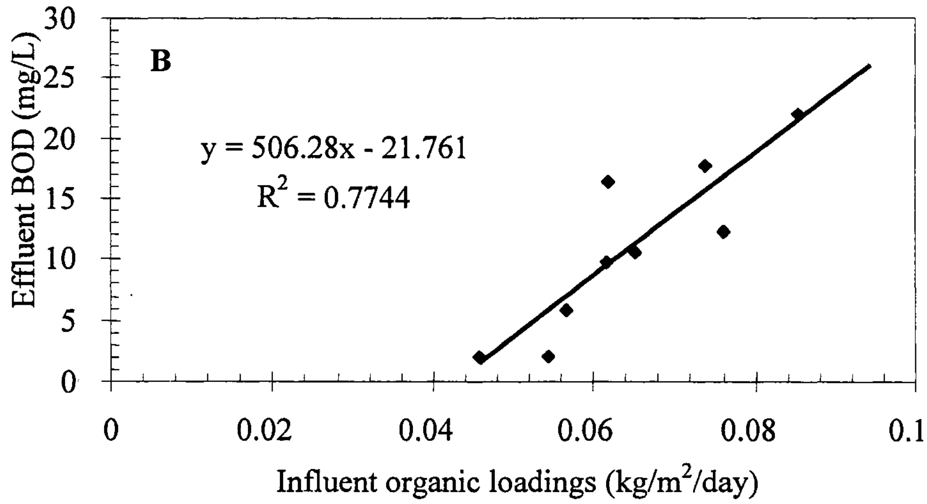


Figure 7.5 First-order model of BOD_5 removal with influent organic loadings by the dual media RBF with the geotextile volume percentage of 25%, including under the operation of A) low HLR; and B) high HLR.

Figure 7.5 shows the first-order model for dual media RBFs dosed at a low HLR (Figure 7.5.A) and a high HLR (Figure 7.5.B). These first-order models suggest that the dual media RBF with the geotextile volume percentage of 25 % could produce an effluent with low BOD_5 under organic loading rates that are higher than conventionally designed recirculating sand filters.

Nitrification

The nitrification process was evaluated by quantifying the concentrations of NH_4^+-N and NO_3^--N in the RBF influent and effluent samples. Water quality entering the filter beds (i.e., influent) was represented by the measured water quality in the recirculation tank effluent samples. Thus, NH_4^+-N and NO_3^--N decrease/increase rates within filter beds were determined by the discrepancy between the water quality leaving recirculation tanks and filter beds. In this chapter, the occurrence of nitrification was supported by the observed decreased of NH_4^+-N, and the increase of $NO3^-$-N in the filter effluents as compared to the filter influents (the measured recirculation tank effluents). The following demonstration in this section provided the detailed discussion of nitrification process occurred within the filter bed.

I. NH_4^+-N Removal

Single media RBFs. The 100 % crushed glass RBFs produced effluents with very low NH_4^+-N concentrations at both low and high HLRs as shown in Figure 7.6.A and B. The average NH_4^+-N concentrations were 0.8 ± 0.5 and 0.4 ± 0.5 mg/ L at the low and high HLRs, respectively. In contrast, the 100 % geotextile RBFs dosed at the high HLR showed a much higher effluent NH_4^+-N concentrations as compared to at the low HLR. The average effluent NH_4^+-N concentrations for these two geotextile RBFs were 0.8 ± 0.5 and 12.8 ± 11.9 mg/ L at the low and the high HLRs, respectively (Figure 7.6.A and B). An ANOVA analysis showed that there was a significant difference amongst these four single media RBFs in terms of NH_4^+-N removal (α =0.05), as shown in Appendix: J – ANOVA – Single Media – NH_4^+-N.

Dual media RBFs. Figure 7.6.A shows that at the low HLR, the average effluent NH_4^+-N concentrations were 0.3 ± 0.5, 0.3 ± 0.3 and 7.0 ± 4.2 mg/ L for the dual RBFs with the geotextile volume percentage of 25, 50 and 75 %, respectively. Figure 7.6.B shows that at the high HLR, the average effluent NH_4^+-N concentrations were 0.7 ± 0.7, 12.7 ± 9.7 and 11.5 ± 5.6 mg/ L for the dual media RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively. These data demonstrate that the RBF effluent NH_4^+-N concentrations were highly variable amongst these six dual-media RBFs. ANOVA results showed that there was a significant difference amongst these six dual media RBFs for the NH_4^+-N removal (α=0.05), as shown in Appendix: J – ANOVA – Dual Media – NH_4^+-N.

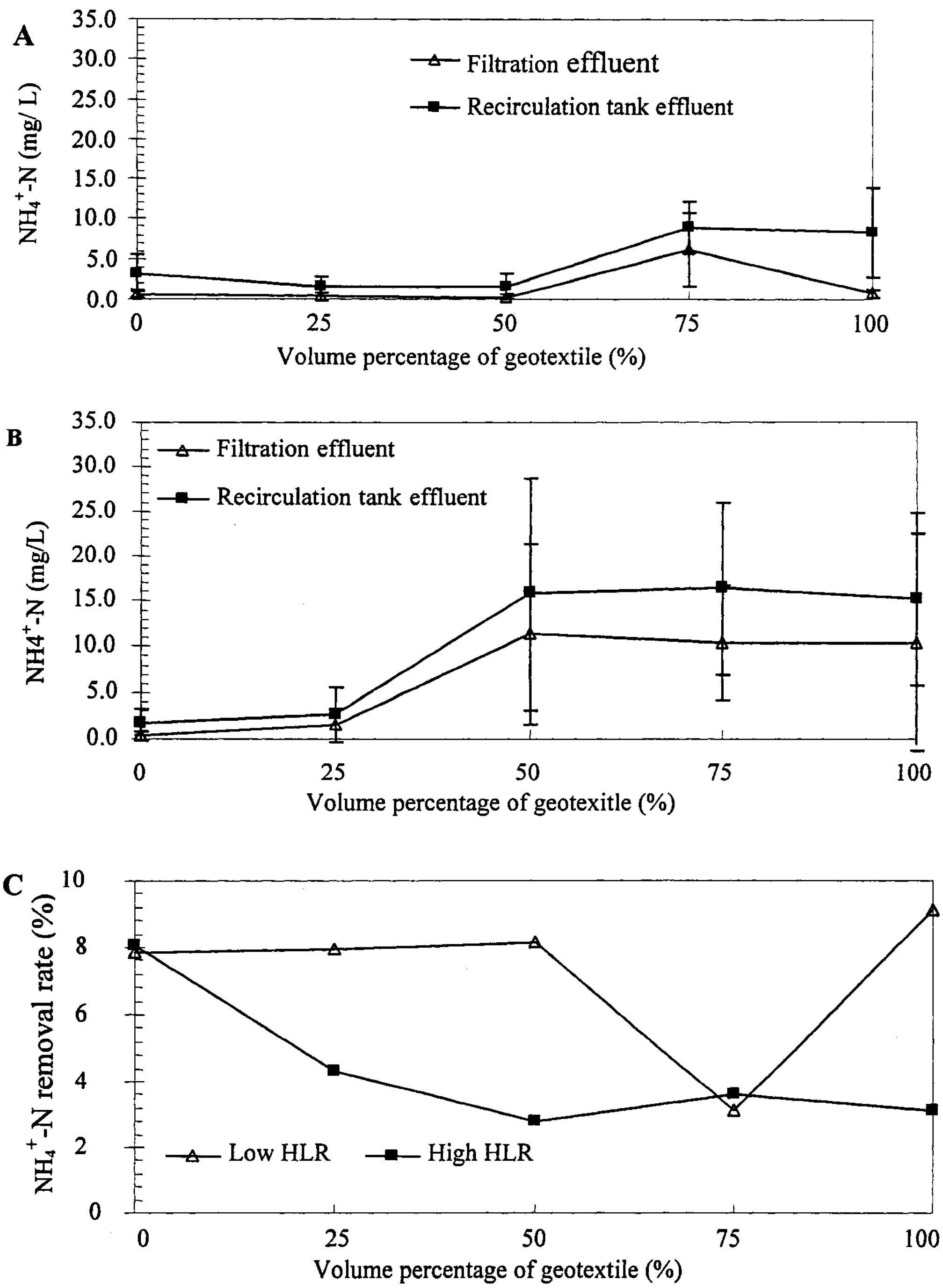


Figure 7.6 NH_4^+-N concentrations in the biofiltration and the recirculation tank effluents of RBFs with various geotextile volume percentages (error bars indicate standard deviation from mean), including A) low HLR; B)high HLR; and C) NH_4^+-N removal rates within filtration beds.

Percentage of Geotextile. For the RBFs dosed at the low HLR, Figure 7.6.A shows that a high geotextile volume percentage generally resulted in a high effluent NH_4^+-N. At the higher HLR (i.e., 0.40 $m^3/m^2/d$), the 100 % geotextile RBF produced an effluent with higher NH_4^+-N concentrations as compared to the effluent from the 100 % crushed glass RBF. These results might have been due to the lower hydraulic retention time within the filter bed of the geotextile RBF as compared to the crushed glass RBF, since the porosity of the geotextile was much higher (e.g., 0.90) than that of the crushed glass (e.g., 0.36) in this study. This assumption is consistent with USEPA (2002) that nitrification normally occurs after depletion of BOD_5 concentrations has occurred. Therefore, it is hypothesized that lower hydraulic retention times within the geotextile filter bed dosed at the high HLR could not effectively support the growth of nitrifying bacteria before the completion of BOD_5 removal.

It was observed in Figure 7.6.A that the RBF effluent NH_4^+-N concentrations corresponded with the recirculation tank effluent NH_4^+-N concentrations for all of the RBFs tested in this study. Specifically, a high recirculation tank effluent NH_4^+-N concentration consistently resulted in a high RBF effluent NH_4^+-N concentration. Since the recirculation tank effluent NH_4^+-N concentrations of the dual media RBF with the geotextile volume percentage of 75% was higher than those of the other four RBFs, the RBF effluent NH_4^+-N concentration of this particular RBF was accordingly higher than others as shown in Figure 7.6.A. Consequently, the NH_4^+-N removal rate of the RBF with geotextile volume percentage of 75 % was lower than NH_4^+-N removals found in the other RBFs, as shown in Figure 7.6.C. It was also observed that NH_4^+-N removals within the filter beds of the other four RBFs were quite consistent, ranging approximately from 79 to 92 % as shown in Figure 7.6.C.

For the RBFs dosed at the high HLR, Figure 7.6.C shows that the 100 % crushed glass RBF provided the highest NH_4^+-N removal as compared to the other four RBFs. Specifically, NH_4^+-N concentrations were reduced by 81 % in the 100 % crushed glass filter based on the RBF effluent and the recirculation tank effluent NH_4^+-N concentrations. However, the other four RBFs provided less than 50 % NH_4^+-N removal within filter beds at the higher HLR evaluated in this study.

II NO$_3^-$-N Removal

As previously discussed in Section 5.4.2.2, nitrate (NO_3^--N) is produced during the nitrification process. In order to determine the level of nitrification within the bench-scale RBF systems, NO_3^--N was measured in samples taken from both the recirculation tank and the filtration effluents.

Single media RBFs. The 100 % crushed glass RBF dosed at the low HLR produced an effluent with higher NO_3^--N concentrations as compared to the recirculation tank effluent as shown in Figure 7.7.A. The average NO_3^--N concentrations for this RBF at the low HLRs were 3.5 ± 1.9 mg/ L and 12.8 ± 4.1 mg/ L in the recirculation tank and the biofiltration effluents, respectively (Figure 7.7.A). For the 100 % crushed glass RBF dosed at the high HLR, however, Figure 7.7.B shows that very limited NO_3^--N was generated within the filter bed, since the average NO_3^--N concentrations were 9.5 ± 4.8 mg/L and 9.8 ± 5.9 mg/L in the recirculation tank and the filtration effluents, respectively.

The two 100 % geotextile RBFs dosed at the low and high HLRs showed a similar trend in NO_3^--N generation within the filter bed as that of two crushed glass RBFs. As shown in Figure 7.7.A and B, the geotextile RBF dosed at the low HLR generated more NO_3^--N than the geotextile RBF dosed at the high HLR. The average NO_3^--N concentrations were 1.8 ± 2.0 mg/ L and 15.8 ± 4.5 mg/ L in the recirculation tank and the biofiltration effluents of the geotextile RBF dosed at the low HLR, respectively (Figure 7.7.A). For the geotextile RBF dosed at the high HLR, the average effluent NO_3^--N concentrations in the recirculation tank and the biofiltration effluents were 4.3 ± 4.1 mg/ L and 8.6 ± 4.7 mg/ L, respectively (Figure 7.7. B).

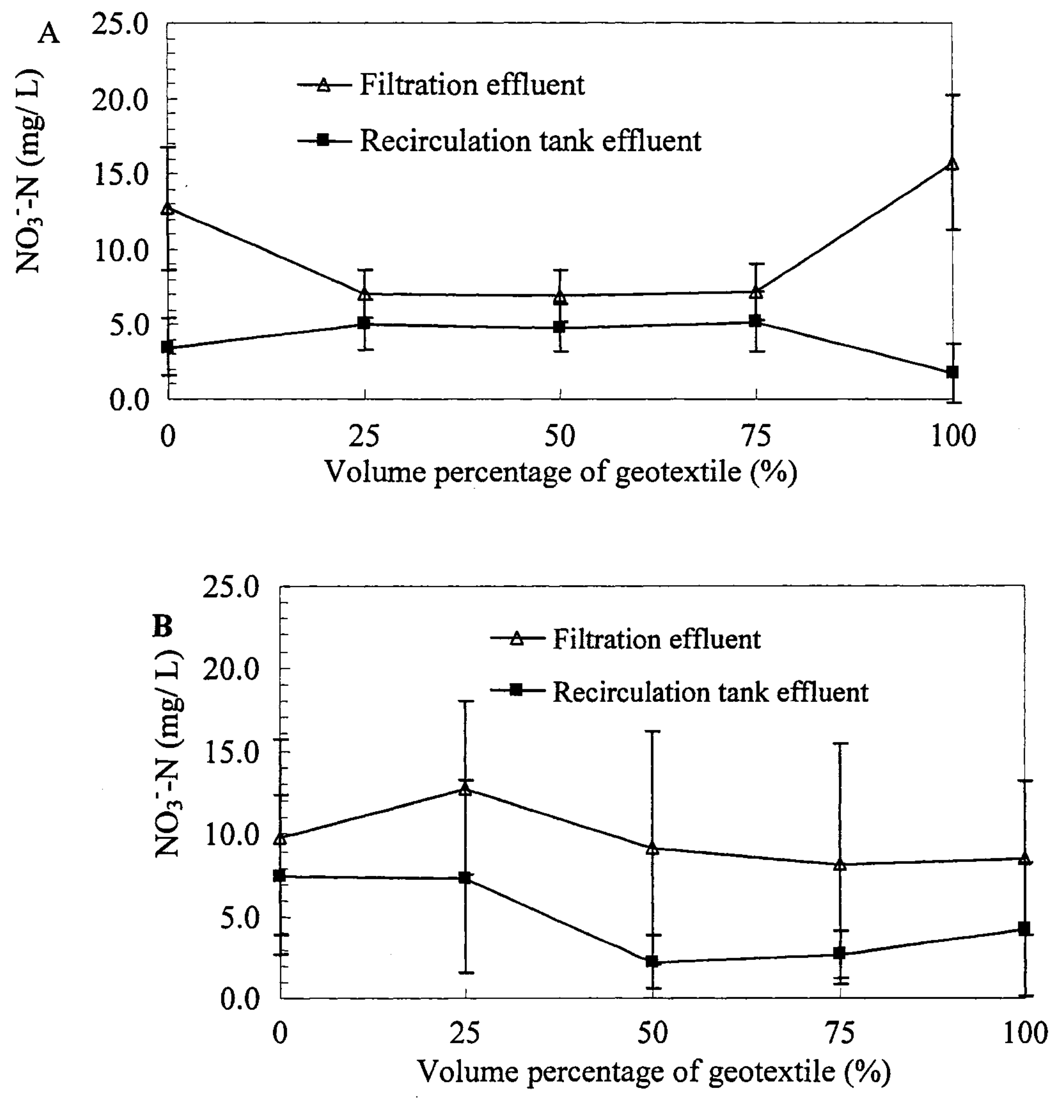


Figure 7. 7 Effluent NO_3^--N concentration at A) low HLR and B) high HLR *(Error bars indicate standard deviation from mean)*

Dual media RBFs. Figure 7.7.A shows that the RBF effluent NO_3^--N concentrations of three dual RBFs dosed at the low HLR were very similar, including 7.0 ± 1.6 mg/ L, 6.9 ± 1.8 mg/ L, and 7.2 ± 1.9 mg/ L for the geotextile volume percentages of 25, 50 and 75 %, respectively. It was also observed in Figure 7.7.A that limited nitrification occurred within the filter beds of the dual media RBFs dosed at the low HLR. The NO_3^--N concentrations in the recirculation tank effluents were 5.0 ± 1.7 mg/ L, 4.8 ±

1.6 mg/ L, and 5.2 ± 2.0 mg/ L of the RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively.

For the dual media RBFs dosed at the high HLR, Figure 7.7.B shows that the effluent NO_3^--N concentrations decreased with the increasing percentages of geotextile within the filter beds. The average biofiltration effluent NO_3^--N concentrations for the three dual media RBFs were 12.8 ± 5.2 mg/ L, 9.2 ± 7.1 mg/ L, and 8.2 ± 7.3 mg/ L for the geotexitle volume percentages of 25, 50 and 75 %, respectively. In addition, more NO_3^--N generation was observed within filter beds of the dual media RBFs dosed at the high HLR than that of the dual media RBFs dosed at the low HLR. The average NO_3^--N concentrations of the recirculation tanks effluents were 7.4 ± 5.8 mg/ L, 2.2 ± 1.7 mg/ L, and 2.7 ± 1.5 mg/ L for the geotextile volume percentages of 25, 50 and 75 %, respectively.

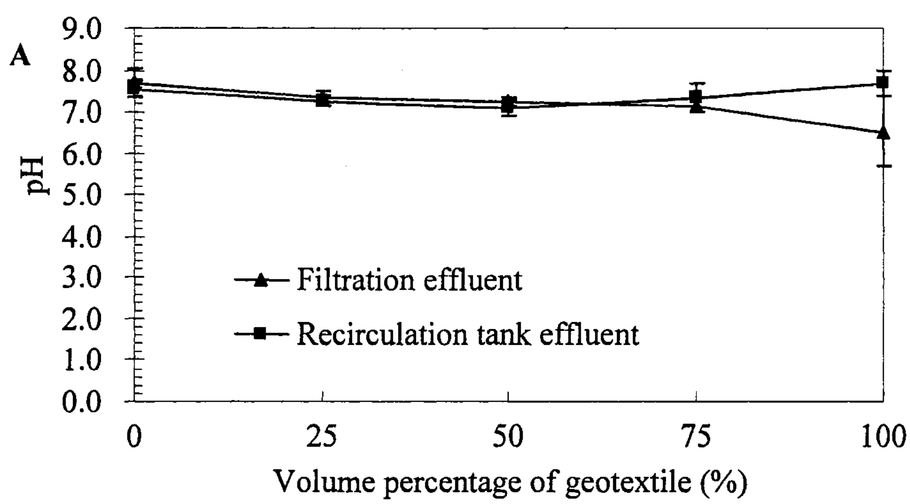
III pH Variation

As presented in Equation 5.1 in Section 5.4.2.2, one mol of NH_4^+-N can generate 1.97 mol of H^+ during the nitrification process. Therefore, a decrease in wastewater pH can be used as an indicator of nitrification (Rittmann and McCarty, 2002). As such, pH was measured in samples taken from the recirculation tank and the filter effluents.

Single media RBFs. Samples taken from the 100 % crushed glass RBF systems showed very similar pH levels in the recirculation tank and the biofiltration effluents under the operation of both low and high HLRs as shown in Figure 7.8.A and B. The average pH values at the low HLR were 7.5 ± 0.2 and 7.7 ± 0.3 in the recirculation tank and the biofiltration effluents, respectively (Figure 7.8.A). The average pH values at the high HLR were 7.0 ± 0.2 and 6.9 ± 0.3 in the recirculation tank and the filtration effluents, respectively (Figure 7.8.B).

The 100 % geotextile RBF dosed at the low HLR showed a decrease in pH values from an average of 7.7 ± 0.3 in the recirculation tank effluent to 6.5 ± 0.9 in the biofiltration effluent as shown in Figure 7.8.A. Figure 7.8.B shows a slight decrease of pH from 7.8 ± 0.2 in the recirculation tank effluent to 7.4 ± 0.3 in the biofiltration effluent when the geotextile RBF was dosed at the high HLR.

Dual media RBFs. For the dual media RBFs dosed at the low HLR, Figure 7.8.A shows that the biofiltration effluent pH concentrations were very similar, including 7.3 ± 0.2, 7.3 ± 0.1, and 7.2 ± 0.2 for the RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively. The average effluent pH from samples taken from the recirculation tanks of the dual media RBFs were 7.3 ± 0.1, 7.1 ± 0.2, and 7.4 ± 0.3 for the geotextile volume percentages of 25, 50 and 75 %, respectively. Therefore, no evident pH differences between the recirculation tank and the biofiltration effluents were observed for the dual media RBFs operated at the low HLR.



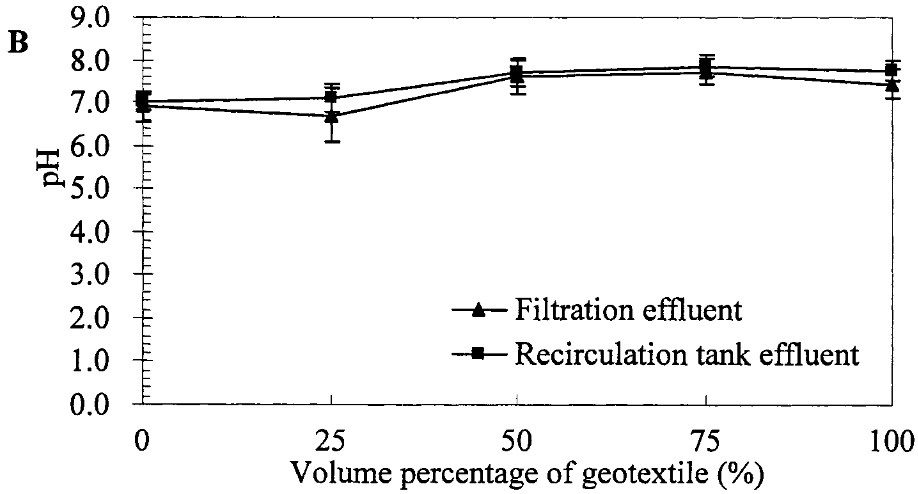


Figure 7. 8 pH in RBF and recirculation tank effluents at A) low HLR and B) high HLR. *(Error bars indicate standard deviation from mean)*

For the dual media RBFs dosed at the high HLR, Figure 7.8.B shows that the effluent pH values in the biofiltration effluents were slightly lower than in the recirculation tank effluents. The overall average biofiltration effluent pH values were 6.7 ± 0.6, 7.6 ± 0.4, and 7.7 ± 0.3 for these three dual media RBFs with the geotexitle volume percentages of 25, 50 and 75 %, respectively. The overall average pH concentrations in the recirculation tank effluents for these three dual media RBFs were 7.1 ± 0.3, 7.7 ± 0.3, and 7.9 ± 0.3 for the geotextile volume percentages of 25, 50 and 75 %, respectively.

Based on all the observed data of NH_4^+-N, NO_3^--N, and pH in the present study, this Chapter found that nitrification occurred within the filter beds due to the decrease of NH_4^+-N and the increase of NO_3^--N within the filter beds. However, this study did not found an apparent decrease of pH in the biofiltration effluents as compared to the recirculation tank effluents.

Pathogen Microorganism Removal

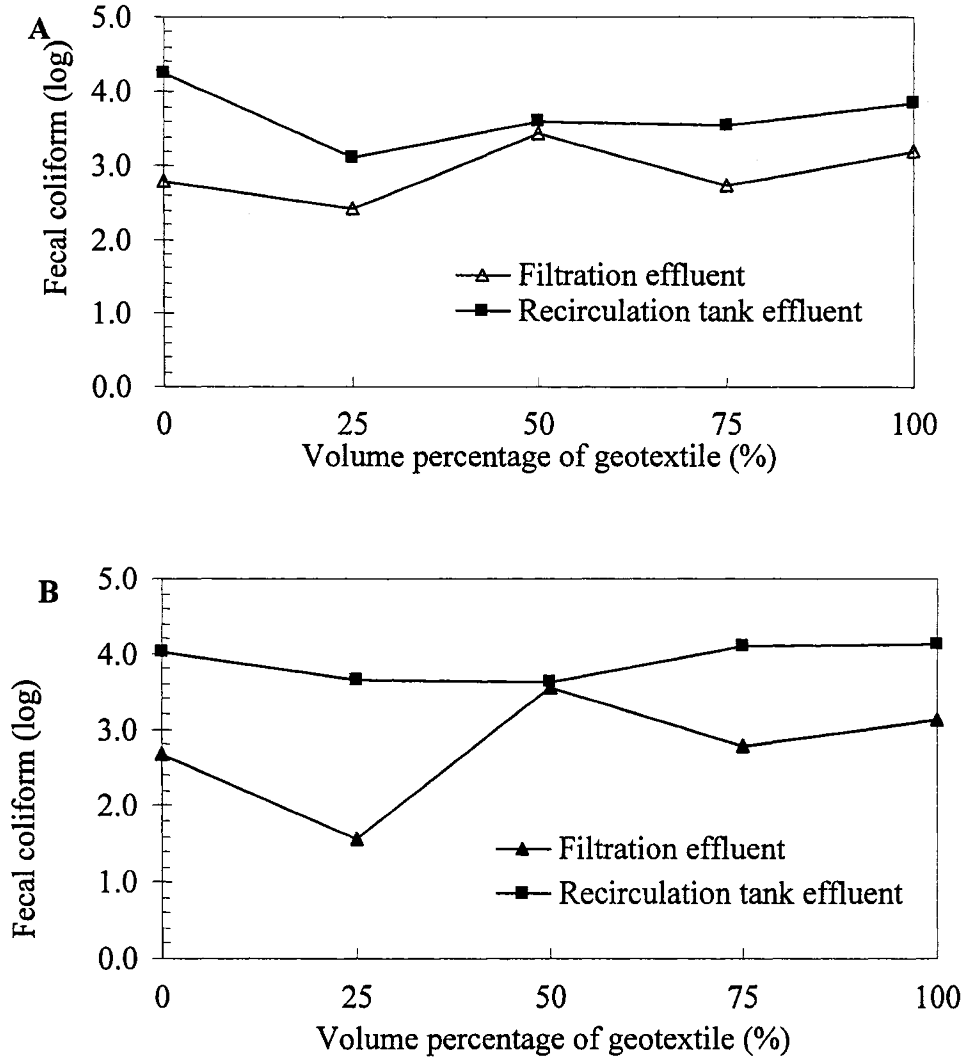


Figure 7. 9 Fecal coliforms in RBF and recirculation tank at A) low HLR and B) high HLR. *(Error bars indicate standard deviation from mean)*

Single media RBFs. Figure 7.9.A and B shows that the average effluent fecal coliform log-reductions were 2.8 and 2.7 for the crushed glass RBFs dosed at the low and high HLRs, respectively. The average effluent fecal coliform log-reductions were 3.2 and 3.1 for the geotextile RBFs dosed at the low and high HLRs, respectively (Figure 7.9.A and B). The results of an ANOVA analysis showed that there was no significant difference amongst these four single media RBFs in terms of fecal coliform log reductions at a confidence level of 95%, as shown in Appendix: J – ANOVA – Single Media – Fecal coliforms.

Dual media RBFs. It was observed from Figure 7.9.A and B that effluent fecal coliform log-reduction varied over a broad range amongst the six dual-media RBFs. For the dual media RBFs dosed at the low HLR, the average log-reduction of fecal coliform was 2.4, 3.5 and 2.7 for the RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively. For the dual media RBFs dosed at the high HLR, the average log-reduction of fecal coliform were 1.6, 3.5 and 2.8 for the RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively.

7.4.2 Chemical Reaction within the Recirculation Tank

According to the observation and analysis discussed in Chapter 5 and 6, three chemical reactions are hypothesized to have occurred in the recirculation tank, including TN removal, NH_4^+-N removal and BOD_5 biodegradation. Similarly, this chapter also discusses the chemical reactions occurring in the recirculation tank to determine contaminant removal in the recirculation tank.

Denitrification

The occurrence of denitrification in the recirculation tank was supported by the decrease of both TN and NO_3^--N. Figure 7.10 presents the projected and measured TN in the recirculation tank, and the measured TN in the filtration effluent. The projected TN in the recirculation tank was determined by the measured septic tank and the filtration effluents TN concentrations as described in Equation 5.1. As presented in Figure 7.10, there was a discrepancy between the projected and the actual measured TN concentrations in the recirculation tanks of most of the RBFs in this chapter, indicating that total nitrogen removal occurred in the recirculation tanks.

I. TN Removal

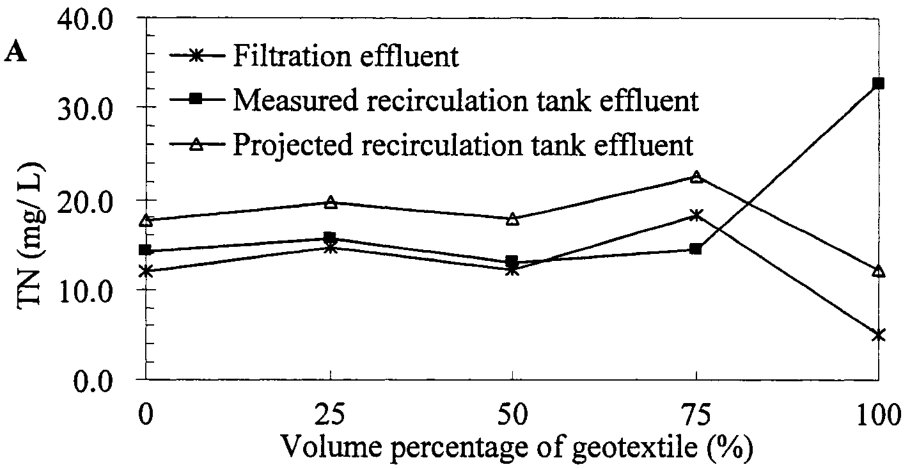
Single Media RBFs. In the crushed glass RBF dosed at the low HLR, the average projected TN into and the actual measured TN leaving the recirculation tank were 17.6 mg/ L and 14.3 mg/ L, respectively as shown in Figure 7.10.A. Based on these data, TN was reduced by 19 % in the recirculation tank. In the crushed glass RBF dosed at the high HLR, the average projected TN into and the actual measured TN leaving the recirculation tank were 19.1 mg/ L and 14.9 mg/ L, respectively as shown in Figure 7.10.B. Based on these data, a 22 % reduction of TN in the recirculation tank of the crushed glass RBF dosed with high HLR was achieved.

In the geotextile RBF dosed at the low HLR, the average projected TN into and the actual measured TN leaving the recirculation tank were 12.1 mg/ L and 32.7 mg/ L, respectively as shown in Figure 7.10.A. These data show that TN actually increased instead of being removed in the recirculation tank. This result is not consistent with the findings described in the previous chapters and the results of the crushed glass filter presented above. In the geotextile RBF dosed at the high HLR, the average projected TN into and the actual measured TN leaving the recirculation tank were 25.7 mg/ L and 24.7 mg/ L, respectively as shown in Figure 7.10.B. Based on these data, only 3.8 % or very limited TN was removed in the recirculation tank of the geotextile RBF dosed with high HLR. It was observed in Figure 7.10 that the TN removal in the recirculation tanks of geotextile RBFs were less those of crushed glass RBFs.

Dual Media RBFs. In the dual media RBF with geotextile volume percentage of 25 % dosed at the low HLR, the average projected TN into and the actual measured TN leaving the recirculation tank were 19.6 mg/ L and 15.7 mg/ L, respectively as shown in Figure 7.10.A. Based on these data, TN was reduced by 20.2 % in the recirculation tank of the dual media RBF with the geotextile volume percentage of 25 %, when it was dosed at the low HLR. A TN reduction of 27 % was achieved in the recirculation tank of the dual media RBF with the geotexitle volume percentage of 50 %, when it was dosed with low HLR. In particularly, the average projected TN into and the actual measured TN leaving the recirculation tank were 17.8 mg/ L and 13.0 mg/ L, respectively as shown in Figure 7.10.A. It was also observed that the average projected TN into and the actual measured TN leaving the recirculation tank of the dual media RBF with geotextile volume percentage of 75 % were 22.6 mg/ L and 14.4 mg/ L, respectively as shown in Figure 7.10.A. Based on these results, a TN reduction of 36 % was achieved in the recirculation tank. Figure 7.10.A shows that increasing the volume percentage of geotextile in the RBF resulted in higher TN reductions in the recirculation tank when the bench-scale RBFs were dosed at the low HLR.

In the dual media RBF with the geotextile volume percentage of 25 %, the average projected TN into and the actual measured TN leaving the recirculation tank were 19.1 mg/ L and 14.9 mg/ L, respectively, when it was dosed at the high HLR as shown in Figure 7.10.B. Based on these data, a TN reduction of 22.0 % was achieved in the recirculation tank. However, a very limited TN removal was observed for the dual media RBFs with the geotextile volume percentages of 50 and 75 % under the operation at the high HLR as presented in Figure 7.10.B. In Particular, in the dual media RBF with the geotextile volume percentage of 50 % under the operation at the high HLR, the average projected TN into and the actual measured TN leaving the recirculation tank were 30.1 mg/ L and 30.0 mg/ L, respectively. In the dual media RBF with the geotextile volume percentage of 75 % under the operation at the high HLR, the average projected TN into and the actual measured TN leaving the recirculation tank were 23.7 mg/ L and 23.3 mg/ L, respectively.

A



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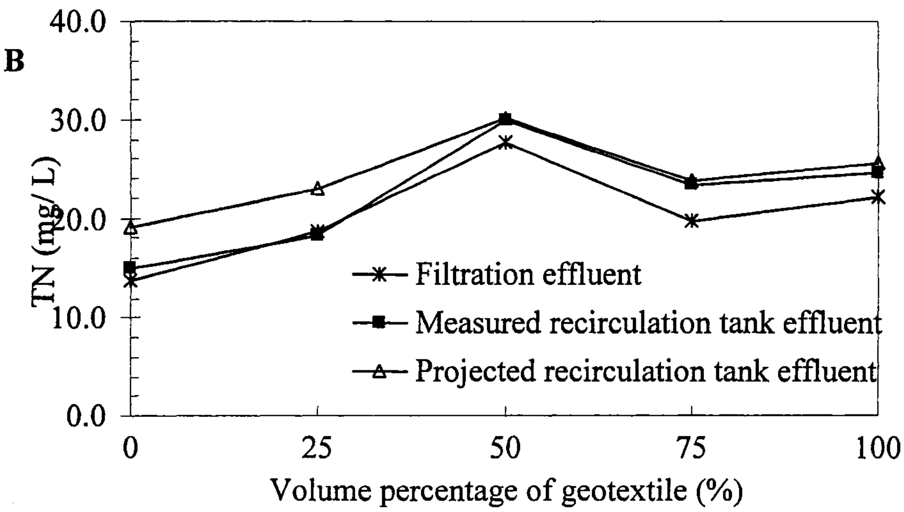


Figure 7.10 Estimation of TN removal between projected and the measured values in the recirculation tank, and the measured filtration effluent: A) low HLR and B) high HLR

Main Reactor for TN Removal. Figure 7.10 shows that there was limited TN reduction within the filter beds of the bench-scale RBFs, since the actual measured recirculation tank effluents TN concentrations were very similar to the measured filtration effluents TN. This observation is consistent with the observations reported in Chapter 5 and 6, and further support the theory that TN removal mainly occurred in the recirculation tank.

II NO_3^--N Removal

To investigate the occurrence of denitrification within the recirculation tanks of the bench-scale RBF systems, NO_3^--N concentrations in the recirculation tank were measured and compared to the projected NO_3^--N concentrations based on Equation 5.1.

Single Media RBFs. In the crushed glass RBF dosed at the low HLR, the actual measured NO_3^--N was 3.5 mg/ L in the recirculation tank. The projected NO_3^--N in the recirculation tank was 10.4 mg/ L as shown in Figure 7.10.A. Therefore, approximately 66 % of the NO_3^--N was removed in the recirculation tank. When the crushed glass RBF was dosed at the high HLR, the projected NO_3^--N into and the actual measured NO_3^--N leaving the recirculation tank were 8.0 and 7.5 mg/ L, respectively as shown in Figure 7.10.B. Based on these data, only 6 % of NO_3^--N reduction was achieved at the higher HLR. These results demonstrate that NO_3^--N removal in the recirculation tank was 10-fold higher in the RBF system operated at the lower HLR.

In the geotextile RBF dosed at the low HLR, the projected NO_3^--N into and the actual measured NO_3^--N leaving the recirculation tank were 12.8 and 1.8 mg/ L, respectively as shown in Figure 7.10.A. Based on these data, approximately 92 % NO_3^--N reduction was achieved in the recirculation tank. When the geotextile RBF was dosed at the high HRL, the projected NO_3^--N into and the actual measured NO_3^--N leaving the recirculation tank were 7.1 and 4.3 mg/ L, respectively as shown in Figure 7.11.B. Based on these data, approximately 39 % NO_3^--N reduction was achieved in the recirculation tank. The results of the geotextile RBFs are similar to those obtained in the crushed glass RBFs trials in that higher NO_3^--N removal was achieved in the recirculation tank of when the RBF systems were operated at a low HLR (i.e., 0.2 m^3/m^2/day) as opposed to operating at the high HLR of 0.40 m3/m2/day.

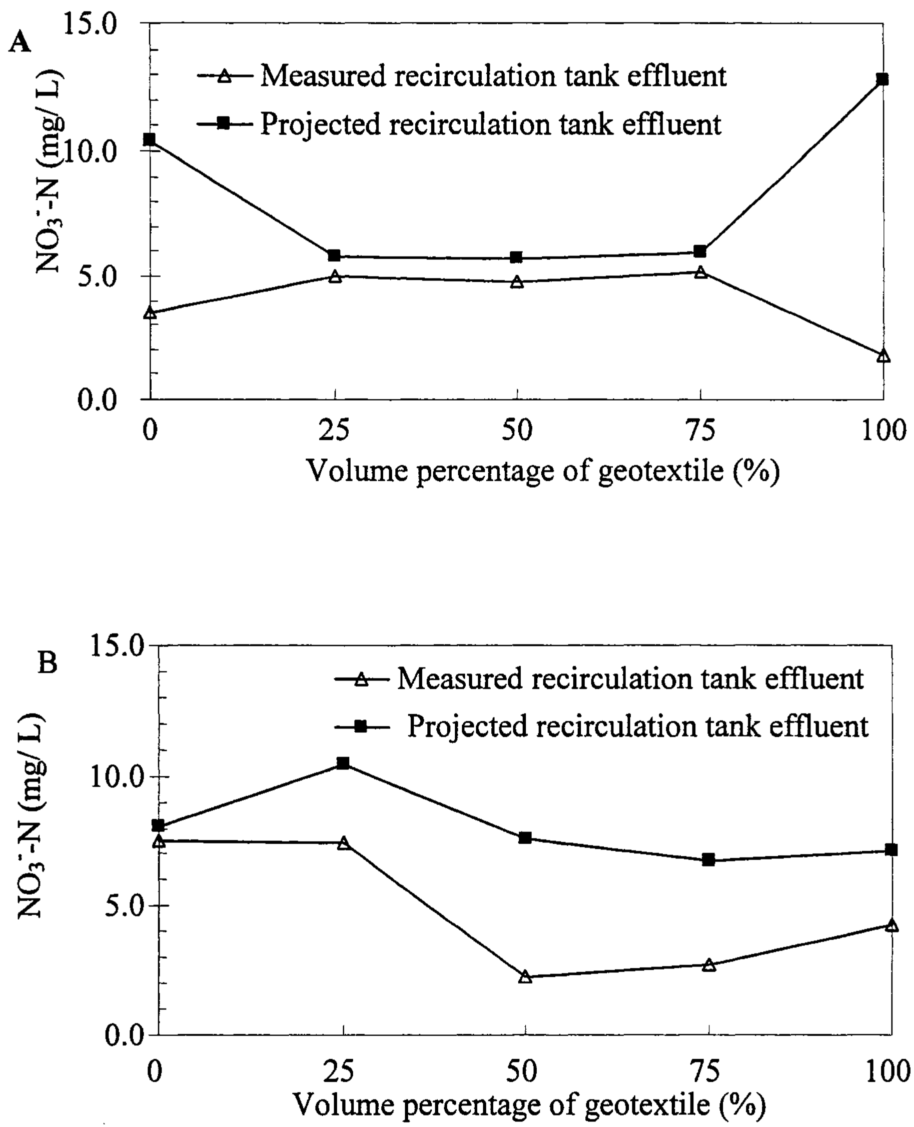


Figure 7.11 NO_3^--N removal in the recirculation tank at the A) low and B) high HLRs.

Dual Media RBFs. When the dual media RBFs were operated at the low HLR, the projected NO_3^--N concentrations into the recirculation tank were 5.8, 5.7 and 5.9 mg/ L for the RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively as shown in Figure 7.11.A. Accordingly, the actual measured NO_3^--N leaving the recirculation tank were 5.0, 4.8 and 5.2 mg/ L for the RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively. Therefore, NO_3^--N reductions of 16, 16 and 12 % were achieved in the recirculation tanks of RBFs with the geotexitle volume percentages of 25, 50 and 75 %, respectively. These data show that the reduction of NO_3^--N in the recirculation tanks were very similar among the three dual media RBFs.

When the dual media RBFs were operated at the high HLR, the projected NO_3^--N concentrations into the recirculation tank were 10.4, 7.5 and 6.7 mg/ L for the RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively as shown in Figure 7.11.B. Accordingly, the actual measured NO_3^--N leaving the recirculation tank were 7.4, 2.2, and 2.7 mg/ L for the RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively. Therefore, the NO_3^--N reduction rates in the recirculation tanks were 29, 71 and 60 % for the dual media RBFs with the geotexitle volume percentages of 25, 50 and 75 %, respectively. These data show that the dual media RBFs operated at the high HLR had a higher NO_3^--N removal in the recirculation tank than that of dual media RBFs operated at the low HLR.

BOD_5 Removal

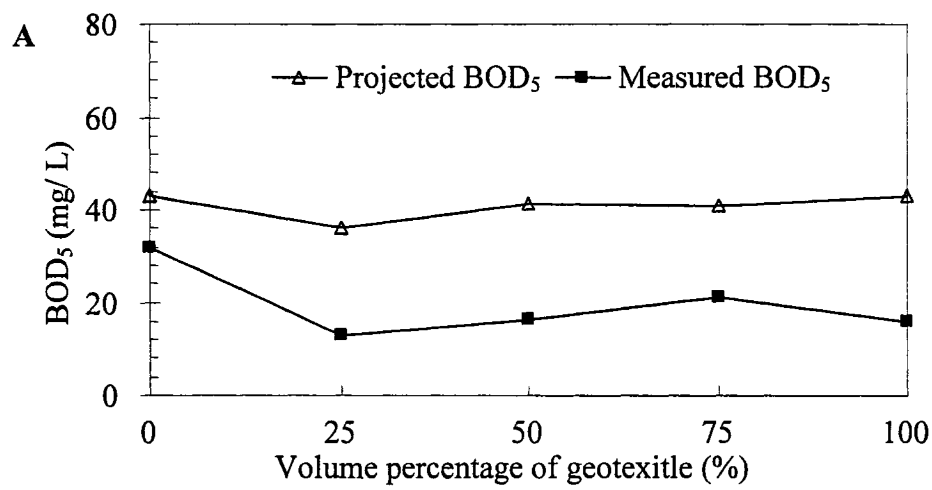
Single Media RBFs. Figure 7.12.A shows that the projected BOD_5 into and the actual measured BOD_5 leaving the recirculation tank of the crushed glass RBF operated at the low HLR was 43 and 32 mg/ L, respectively. Based on these data, a reduction of 26 % BOD_5 was achieved in the recirculation tank. When the crushed glass RBF was operated at the high HLR, the projected BOD_5 into and the actual measured BOD_5 leaving the recirculation tank were 47 and 43 mg/ L, respectively as shown in Figure 7.12.B. These data indicate that approximately 9 % BOD_5 reduction was achieved in the recirculation tank of the crushed glass RBF operated at the high HLR. These results demonstrate that BOD_5 reduction in the recirculation tank of the crushed glass RBF operated at the low HLR was higher than that of the crushed glass RBF operated at the high HLR.

Figure 7.12.A shows that the projected BOD_5 into and the actual measured BOD_5 leaving the recirculation tank of the geotextile RBF operated at the low HLR were 43 and 20 mg/ L, respectively. Based on these data, a reduction of 53 % BOD_5 was achieved in the recirculation tank. When the geotextile RBF was operated at the high HLR, the projected BOD_5 into and the actual measured BOD_5 leaving the recirculation tank were 53 and 38 mg/ L, respectively. Therefore, the BOD_5 reduction in the recirculation tank of the geotextile RBF operated at the high HLR was 28 %. These results demonstrate that BOD_5 reduction in the recirculation tank of the geotextile RBF operated at the low HLR was higher than that of the geotextile RBF operated at the high HLR. These results are also consistent with the observation of the crushed glass RBF in terms of BOD_5 reduction under low and high HLRs.

Dual Media RBFs. When the dual media RBFs were operated at the low HLR, the actual measured BOD_5 leaving the recirculation tank increased with increasing volume percentage of geotextile. Consequently, the BOD_5 reduction in the recirculation tank decreased with increasing volume percentage of geotextile as shown in Figure 7.12.A. In particular, the projected BOD_5 into the recirculation tank were 36, 41 and 41 mg/ L for the dual media RBFs with the volume percentage of geotextile of 25, 50 and 75 % respectively. The actual measured BOD_5 leaving the recirculation tank were 12, 16 and 26 mg/ L for the dual media RBFs with volume percentage of geotextile of 25, 50 and 75 %, respectively. Therefore, the BOD_5 reduction rates in the recirculation tanks were 68, 61 and 37 % for the geotextile volume percentages of 25, 50 and 75 %, respectively.

When the dual media RBFs were operated at the high HLR, the actual measured BOD_5 leaving the recirculation tank increased with increasing volume percentage of geotexitle as shown in Figure 7.12.B. In particular, the measured BOD_5 in the recirculation tanks were 38, 53, and 59 mg/ L for the geotextile volume percentages of 25, 50 and 75 %, respectively. The projected BOD_5 into the recirculation tank were 42, 74, and 67 mg/ L for the geotextile volume percentages of 25, 50 and 75 %, respectively.

Accordingly, the BOD_5 reduction rates in the recirculation tank were 10, 29 and 12 % for the geotextile volume percentages of 25, 50 and 75 %, respectively.



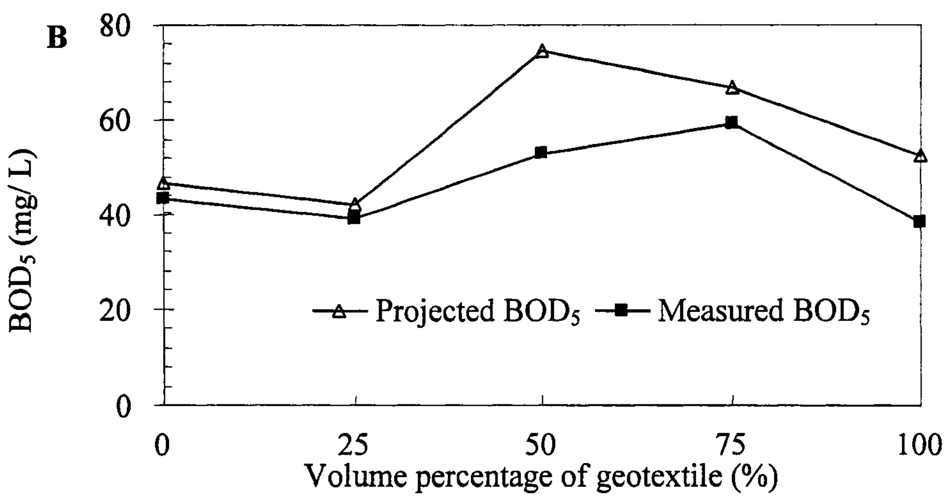


Figure 7.12 Estimation of BOD_5 removal between projected and the measured values in the recirculation tank: A) low HLR and B) high HLR.

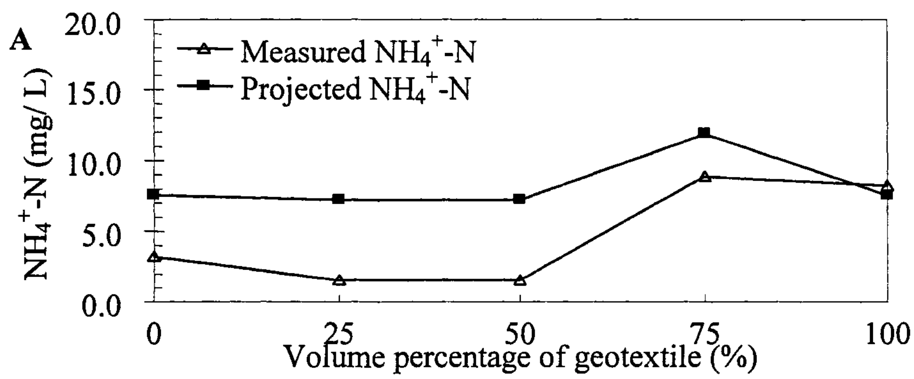
In summary, Figure 7.12 shows that BOD_5 reduction in the recirculation tanks of the dual media RBFs operated at the low HLR were higher than that of dual media RBFs operated at the high HLR. This conclusion is consistent with the observation of the single media RBFs presented above, in that BOD_5 reduction in the recirculation tank of single media RBFs operated at the low HLR was higher than that of the single media RBFs operated at the high HLR.

NH_4^+-N Removal

Single Media RBFs. Figure 7.13.A shows the projected NH_4^+-N and the actual measured NH_4^+-N in the recirculation tank of the crushed glass RBF operated at the low HLR. It was observed that NH_4^+-N removal occurred in the recirculation tank, since the projected and the actual measured NH_4^+-N were 7.6 and 3.2 mg/ L, respectively. Based on these data, a 58 % reduction in NH_4^+-N concentrations was achieved in the recirculation tank. When the crushed glass RBF was operated at the high HLR, the average projected NH_4^+-N into and the actual measured NH_4^+-N concentrations leaving the recirculation tank were 7.3 and 1.8 mg/ L, respectively. Therefore, the reduction of NH_4^+-N in the recirculation tank was 75 %. These data indicate that higher NH_4^+-N removal occurred in the recirculation tank of the crushed glass RBFs operated at the high HLR as compared to the crushed glass RBF system operated at the low HLR.

The geotextile RBF system operated at the low HLR did not show NH_4^+-N removal in the recirculation tank, with the projected and the actual measured NH_4^+-N of 7.6 and 8.3 mg/ L, respectively as shown in Figure 7.13.A. When this system was operated at the high HLR, the projected NH_4^+-N into and the actual measured NH_4^+-N leaving the recirculation tank were 15.4 and 15.3 mg/ L, respectively as presented in Figure 7.13.B. These data show that very limited NH_4^+-N removal occurred in the recirculation tank of the geotextile RBF systems operated at low and high HLRs.

Dual Media RBFs. When the dual media RBF systems were operated at the low HLR, NH_4^+-N reductions in the recirculation tanks of RBFs with geotexitle volume percentages of 25, 50 and 75 % were 79, 78 and 25 %, respectively. Given the results of both of the 100 % geotextile RBF systems (i.e., low and high HLRs) in which no NH_4^+-N reduction was observed in the recirculation tanks , the NH_4^+-N reductions observed in the dual media RBF systems present similar results. Specifically, a high volume percentage of geotextile in the bench-scale RBFs resulted in lower NH_4^+-N reductions in the recirculation tank. Figure 7.13.A shows that the projected NH_4^+-N into the recirculation tanks were 7.3, 7.3, and 11.9 mg/ L for RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively. Accordingly, the actual measured NH_4^+-N were 1.5, 1.6, and 8.9 mg/ L in the recirculation tanks for the dual media RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively.



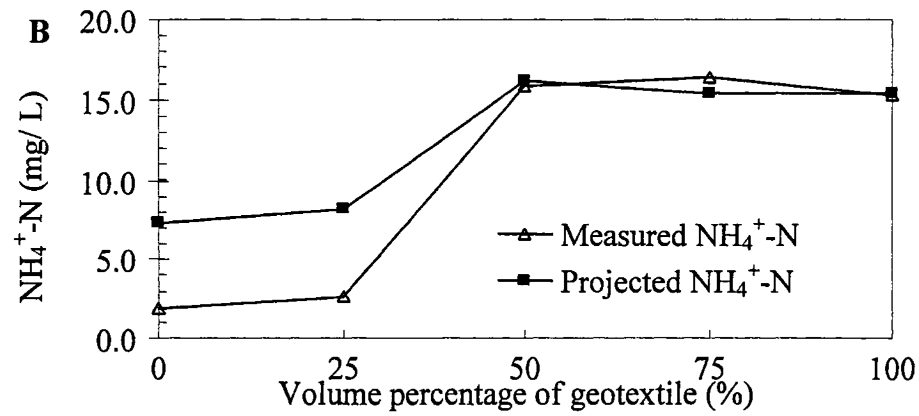


Figure 7.13 Estimation of NH_4^+-N removal between the projected and the measured values in the recirculation tank: A) low HLR and B) high HLR.

When the dual media RBFs dosed at the high HLR, the projected NH_4^+-N into the recirculation tank were 8.2, 16.2 and 15.4 mg/ L for the geotextile volume percentages of 25, 50 and 75 %, respectively as presented in Figure 7.13.A. The actual measured NH_4^+-N in the recirculation tanks were 2.7, 15.9, and 16.4 mg/ L for the geotextile volume percentages of 25, 50 and 75 %, respectively. Accordingly, the NH_4^+-N reductions were 68%, 2%, and 0% in the recirculation tanks of RBFs with the geotextile volume percentages of 25, 50 and 75 %, respectively. Similar to the results of nitrate removal in the dual media RBF systems, these data show that a high geotextile volume percentage resulted in a low NH_4^+-N reduction in the recirculation tank. These results are also consistent with the NH_4^+-N reductions observed in the dual media RBFs operated at the low HLR.

7.4.3 Mass Balance Analysis

A mass balance analysis was conducted to determine substrate removal rates and kinetics coefficients in the recirculation tanks of the bench-scale RBFs. A mass balance of the dual media RBF operated at the high HLR with the geotextile volume percentage of 25 % was presented in Figure 7.14. Appendix: K – Process Overview shows the process overview with mass balance of other nine RBFs tested in this chapter. The detailed description of the mass balance analysis used for the RBF systems has been presented in Chapter 6, and provides the basis of the current analysis of system dynamics in the bench-scale recirculation tanks.

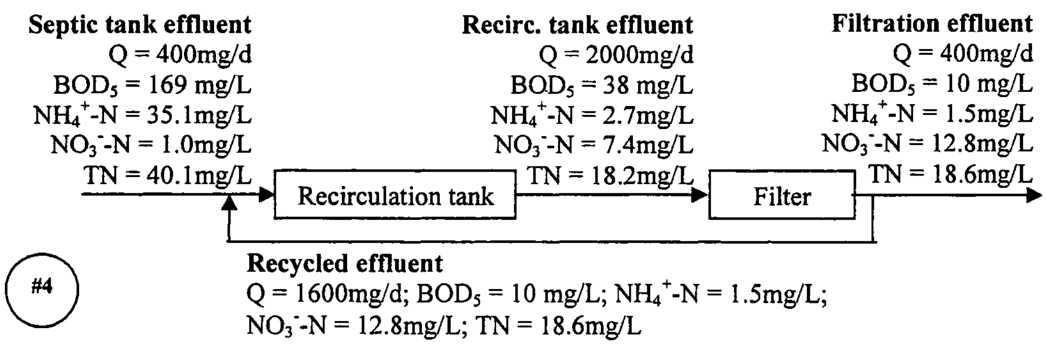


Figure 7. 14 Process overview of bench-scale RBFs (#4 is listed in Table 7.1)

Based on water quality and flow rates presented in Figure 7.14, this chapter determined substrate accumulation rates (biodegradation or generation) in the recirculation tank. The substrates include BOD_5, NH_4^+-N, NO_3^--N and TN. Table 7.1 summaries the accumulation rates of these water quality parameters.

Table 7.1 Summary of substrate decay (+) or accumulation (-) coefficient, day^{-1}.

RBF System	Geotextile volume %	HLR	BOD_5	NH_4^+-N	NO_3^--N	TN
#1	0	Low	0.30	0.86	1.09	0.21
#2	0	High	0.08	1.39	0.07	0.25
#3	25	Low	1.13	1.57	0.15	0.23
#4	25	High	0.10	1.13	0.34	0.23
#5	50	Low	0.93	1.53	0.18	0.31
#6	50	High	0.34	0.02	1.23	0.10
#7	75	Low	0.47	0.29	0.14	0.45
#8	75	High	0.13	-0.06	0.91	0.02
#9	100	Low	0.79	-0.08	1.99	-0.99
#10	100	High	0.32	0.01	0.51	0.04

Process overview of RBF System # 4 is shown in Figure 7.14; The process overview of other nine RBF systems are shown in Appendix K.

It was observed that the coefficients reported in Table 7.1 were highly variable amongst the ten dual media RBFs tests. In particular, the coefficient for BOD_5 decay varied from the minimum value of 0.08 day^{-1} to the maximum value of 1.13 day^{-1}, with the average value of 0.46 day^{-1}. In addition, the range of BOD_5 decay coefficient included the BOD_5 decay coefficient (0.80 day^{-1}) determined from the field-scale RBF study as described in Chapter 6. It was also observed that the decay coefficient for NH_4^+-N in the recirculation tank was from the minimum value of -0.08 day^{-1} to the maximum value of 1.57 day^{-1}. This range also included the reported NH_4^+-N decay coefficient of 0.32 day^{-1} found in the field-scale RBFs study as described in Chapter 6. The NO_3^--N decay coefficient determined in current Chapter 7 varied from the minimum value of 0.07 day^{-1} to the maximum value of 1.99 day^{-1}. Table 7.1 shows that the determined TN decay coefficient in the recirculation tanks of RBFs tested in Chapter 7 were generally lower than that reported in Chapter 6, which was determined based on a field-scale study. Due to the high variation of substrate accumulation coefficients in this study, more research needs to be conducted for further investigation of various chemical and biological reactions in the recirculation tank.

7.4.4 Biofilm Profile in the Dual Media RBF of 3CG:1GT

Scanning Electrical Microscope Images

Scanning electrical microscope (SEM) images were taken to investigate the surface structure of the biofilm formed around the filter media (i.e., crushed glass and geotextile) in the dual media RBF with the geotextile volume percentage of 25 % operated at the low and high HLRs. SEM images were taken at different depths (top of the filter bed was set as 0 cm) of the filter bed including geotextile (0, 2.5 and 3.5 cm) and crushed glass (4.0, 10 and 15 cm), as shown in Figure 7.15. Figure 7.16 presents the biofilm around the geotextile sampled at the depth of 2.5 cm (Figure 7.15), when the RBF was operated at the high HLR. Apparently, two different surface images were observed on the same piece of geotextile as shown in Figure 7.16.B. In particular, Figure 7.16.A shows the dense and rough biofilm attached on the part of the surface of a tested piece of geotextile fiber. However, Figure 7.16.C shows that there was no noticeable biofilm existing on the surface of the other part of the same piece of geotextile fiber.

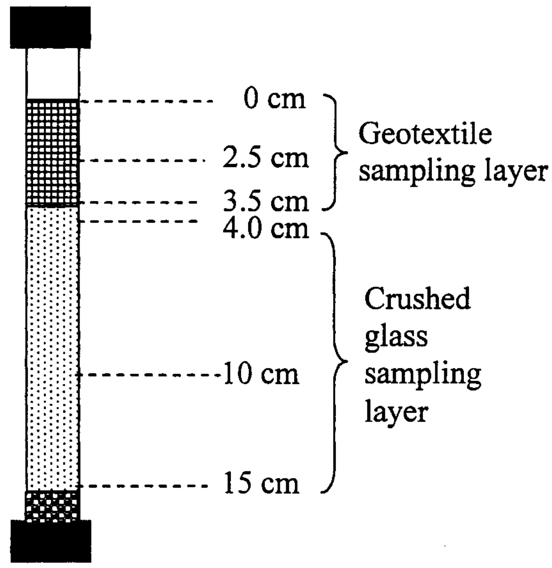


Figure 7.15 Filter media sampling depth for SEM tests

Yaman (2003) provided two conceptual models of biofilm morphology in geotextile biofilters (Figure 7.17.A and B). The first model suggests that biofilm development on the surface of the geotextile fibers defines the pores, or void space between the geotextile fibers (Figure 7.17.A). As presented in Figure 7.17.B, the second model suggests that a biomass of discontinuous floc develops within the void space between the geotextile fibers and has limited contact with the fibers that surround it. Yaman (2003) concluded that the biomass in geotextile biofilters is a combination of the two models presented in Figure 7.17.A and B. This conclusion supports the findings of the current study that showed that biofilm did not exist on all surface areas of the geotextile fibers in the bench-scale biofilter. In another words, this conclusion indicated that geotextile could not provide surface area supporting biomass growth as large as some studies expected (Wren et al., 2004; and Loomis et al., 2004), or only a portion of the geotextile surface was covered by biomass.

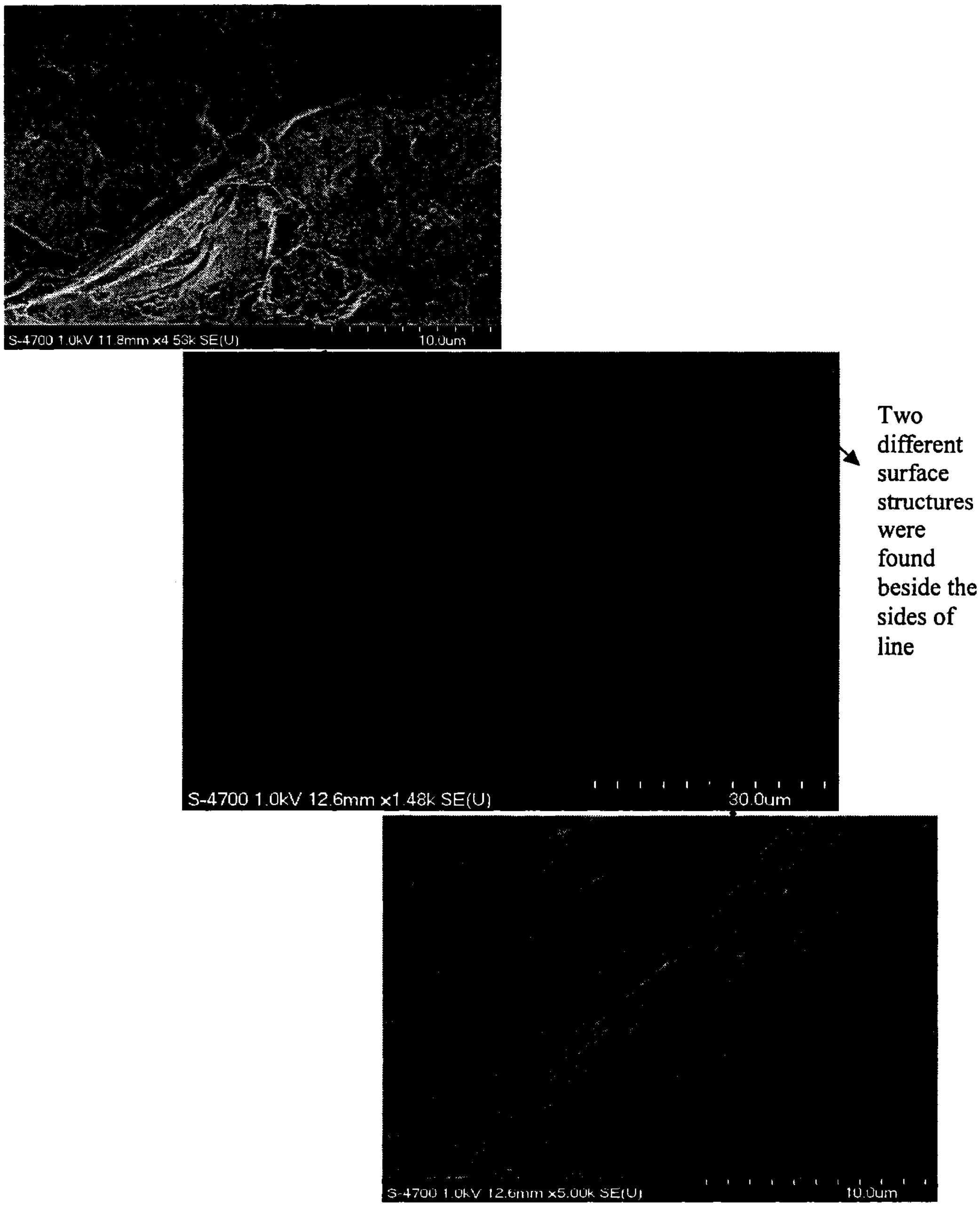


Figure 7.16 Uneven biofilm distribution on the surface of geotextile, including A) biofilm; B) two types of surface structures; and C) geotextile surface structure.

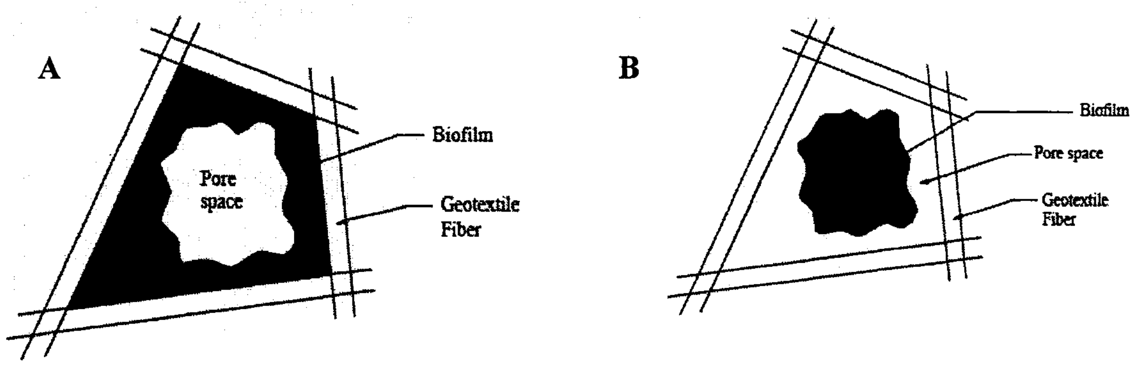


Figure 7.17 Two dimensional biofilm in geotextile filter: A) fiber attached biofilm; B) Floc biomass model. (Source: Yaman, 2003)

Figure 7.18 presents SEM photos of a sample of the crushed glass media of the 25 % geotextile dual media RBF system operated at a low and high HLR. This sample was taken from the bottom layers (i.e., depth of 15 cm in Figure 7.15) of the RBFs. Figure 7.18.A shows that at the low HLR, there was minimal biofilm development on the surface of the crushed glass media. However, a rough and dense biofilm development was observed on the surface of the crushed glass media in the RBF operated at the high HLR. These SEMs suggest that biofilm development in the RBFs was different under the two HLRs evaluated in this study.

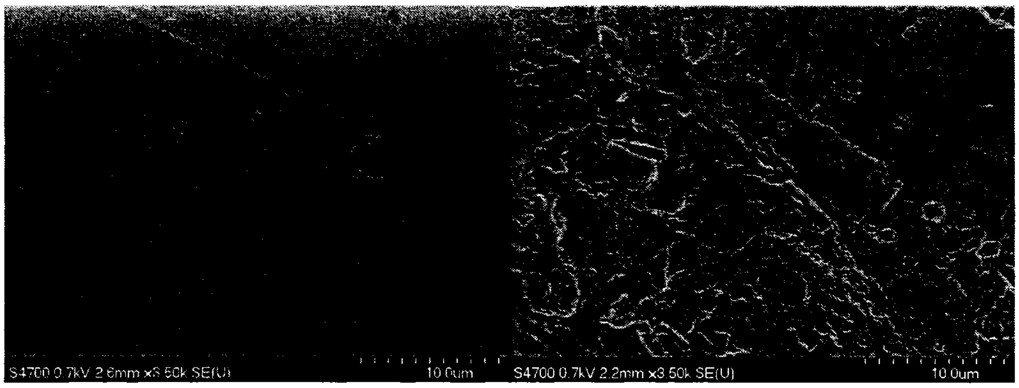


Figure 7.18 Surface structure of biofilm around the crushed glass at the bottom of dual media RBF of 3CG:1GT dosed with two levels of HLRs: A) low HLR; B) high HLR.

Biomass and Filter Media Depth

I Model Introduction

In order to compare biomass growth within and organic loading rates into filter beds, this chapter quantified these two parameters over the entire depth based on a model applied by Leverenz et al. (2000) to investigate the organic matter loading rates into a sand and a geotextile chips RBFs. When this model was applied by Leverenz et al. (2000), no BOD_5 removal in the recirculation tank was included. Therefore, the present Chapter also assumes that BOD_5 removal occurs only within the filter bed, such that the biomass growth and organic loading rates can be simplified and quantified by two equations. Equation 7.1 relates the amount of biofilm to the depth in a packed bed filter and surface area of the medium (Leverenz et al. 2000):

$$\phi_b(d) = SA_m \cdot t_{\max} e^{-k_d d} \qquad [7.1]$$

where

$\phi_b(d)$ = fraction of volume filled by biomass at depth d, %;
SA_m = surface area of the medium, m^2/m^3;
t_{max} = maximum thickness of biofilm (assumed to be 40 μm under ideal conditions);
k_d = distribution coefficient, m^{-1}; and
d = depth in media, m.

The surface layers of a packed bed filter receives the highest concentration of substrate, resulting in the top layer of a packed bed filter having the greatest amount of biomass. As the amount of biomass increases in the surface layers of the packed bed filter, the amount of space available for gas transfer and the infiltration of water is reduced. Media with low porosity can only support a limited amount of biomass before gas transfer and water infiltration is restricted (Leverenz et al., 2000).

Equation 7.2 outlines the relationship between organic loading rate and depth in the medium (Leverenz et al. 2000):

$$O_{\max}(d) = C_{end} \cdot A_b \cdot SW_b \cdot \int_0^d \phi_b(d).dd \qquad [7.2]$$

where

$O_{\max}(d)$ = maximum organic loading rate for filter of depth d, kg BOD_5 /m^2.d;
C_{end} = endogenous respiration coefficient, d^{-1};
A_b = active percent of biomass (assumed to be 5%); and
SW_b = specific weight of biomass, kg/m^3.

For a packed bed filter with a depth, d, the maximum organic loading can be determined using Equation 7.2. If the filter is loaded at a rate higher than the maximum organic loading rate calculated using Equation 7.2, then surface clogging or washout may occur. In a medium with low porosity, it is more likely that surface clogging will occur when overloaded, while in a medium with high porosity, washout is more likely to occur (Leverenz H. et al. 2000).

Packed bed filters are operated in a zero net growth mode. Thus, maximum loading is based on the cell maintenance requirement (no growth) and defined by the endogenous respiration coefficient, C_{end}, equal to 0.05 d^{-1} (Crites and Tchobanoglous, 1998). If the organic loading rate is greater than that required for cell maintenance, the biofilm thickness will increase and fill in more of the void space between the filter media. As the pores narrow, particles bridge and accumulate on the surface of the filter until the rate of wastewater infiltration into the bed is less than the HLR to the filter, resulting in "ponding" of wastewater on the filter surface. As water ponds on the surface of the filter, anaerobic conditions occur in the upper layer of the medium causing odor problems (Leverenz H. et al. 2000).

Table 7.2 Summary of parameters used in model

parameter	unit	Filter Medium	
		Crushed glass	Geotextile
Media surface area (SA_m)	m^2/m^3	1,830	17,000
Media porosity (ε)	%	0.36	0.90
Active biomass (A_b)[1]	%	5	5
Specific weight of biomass (SW_b)[1]	kg/m^3	1,125	1,125
Endogenous coefficient (C_{end})[1]	d^{-1}	0.05	0.05
Distribution coefficient (k_d)[2,3]	m^{-1}	15	10
Maximum biofilm thickness (t_{max})[4]	μm	40	40

[1] Crites and Tchobanoglous, 1998;
[2] Leverenz et al., 2000 for geotextile distribution coefficient;
[3] model fitting parameter for crushed glass with USEPA (2002); and
[4] Tanyolac and Beyenal, 1998.

In this study, equations 7.1 and 7.2 were used to compare the geotextile medium to the crushed glass medium in the bench-scale RBF studies. Table 7.2 summarizes the parameters used in Equation 7.1 and 7.2 to determine the maximum organic loading rates. According to Crites and Tchobanoglous (1998), the specific weight of biomass was set as 1,125 kg/m^3. The distribution coefficient for geotextile was adapted from Leverenz et al. (2000). The distribution coefficient of the crushed glass filter, k_d, was adjusted based on USEPA (2002) recommended values. For multi-pass sand filters the recommended maximum organic loading rate is 0.024 kg BOD/ $m^2 \cdot d$ with the medium effective size of 1 mm (USEPA, 2002). The present study assumes that the distribution coefficient of crushed glass filters is similar as sand filters, since previous chapters have demonstrated that crushed glass performs similarly as sand under identical designs. Based on these experience data, the distribution coefficient for crushed glass was determined to be 15 m^{-1}. Maximum biofilm thickness was determined as 40 μm according to Tayolac and Beyenal (1998).

Figure 7.19 presents the organic loading rate curve for both crushed glass and geotextile based on the model represented by Equations 7.1 and 7.2. The maximum organic loading rate was calculated to be 0.015 kg BOD /$m^2 \cdot d$ for the crushed glass filter bed. This result is consistent with the value reported by Leverenz et al. (2000) that the maximum organic loading rate was 0.016 kg BOD/ $m^2 \cdot d$ for a filter bed containing sand medium.

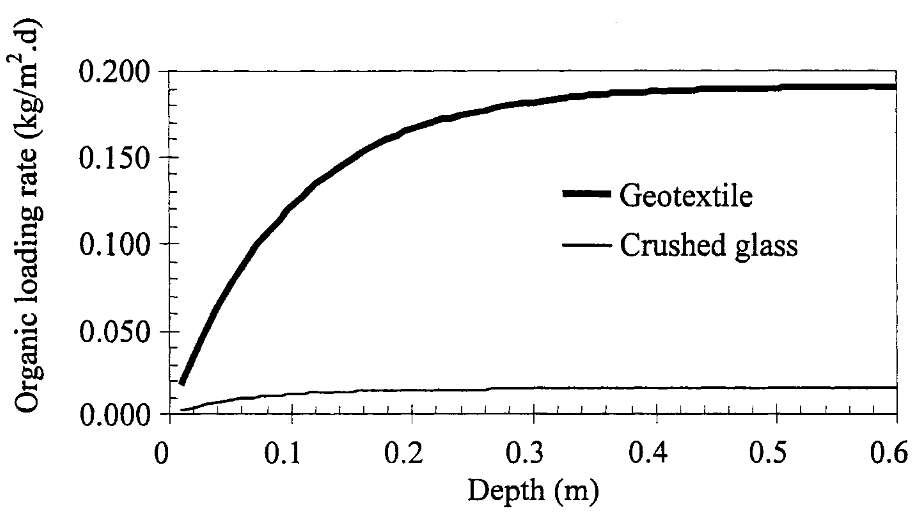


Figure 7.19 Comparison of organic loading rate for a packed bed of crushed glass and geotextile medium

The distribution coefficient of geotextile filter was set as 10 m^{-1} as reported by Leverenz et al. (2000). In their study, geotexitle chips with a porosity of 0.95 were used as the filter medium. This porosity was slightly higher than the porosity of the geotextile fiber used in the current study (e.g., 0.90). Figure 7.19 shows that the maximum organic loading rate was calculated to be 0.191 kg BOD/m^2·d for a packed bed of geotexitle fiber medium. Leverenz et al. (2000) reported that a geotextile chip RBF could be loaded up to 0.161 kg BOD/m^2·d. In addition, Roy and Reid (1998) reported on two textile RBFs for residential and commercial wastewater treatment. For residential wastewater treatment, the organic loading rate and the biofiltration effluent BOD_5 were 0.086 kg BOD/m^2·d and 5 mg/ L, respectively. For commercial wastewater treatment, the organic loading rate and the biofiltration effluent BOD5 were 0.121 kg BOD/m^2·d and 5 mg/ L, respectively. Therefore, the calculated theoretical maximum organic loading of 0.191 kg BOD/m^2·d can be applied to explain the performance of RBFs tested in this chapter, although the calculated maximum organic loading rate was slightly higher than the reported values in literature. Figure 7.19 also indicates that the majority of the organic removal (e.g., 90% removal) is achieved in the upper 0.15 m for the crushed glass filter and in the upper 0.23 m for the geotextile fiber.

Model Application

The organic loading removal rates for single media RBFs were determined by the prediction as presented in Figure 7.19. For dual media RBFs, this study determined the organic loading removal rates at different depths using Equation 7.3:

$$R_d = R_g + (1-R_g)\times R_c \qquad [7.3]$$

where,

R_d = the total organic removal rate of dual media RBFs, %;
R_g = the organic removal rate achieved by geotextile layer, %; and
R_c = the organic removal rate achieved by crushed glass layer, %.

Using Equation 7.3, the theoretical maximum organic loading removal rates were calculated at different depths of a packed bed of crushed glass and geotextile fiber media (Table 7.3).

The actual measured BOD_5 removals in the filter beds shown in Table 7.3 were found to be lower than the theoretically predicted BOD_5 removals, which were the total BOD_5 removal by a complete RBF system. As previously described in Chapter 5 and Chapter 6, BOD_5 removal occurs in both the filter beds and recirculation tanks of RBF systems. The measured BOD_5 removal within the filter bed as shown in Table 7.3 was determined by calculating the difference between the BOD_5 concentrations measured in the recirculation tank and the filter bed effluents. Therefore, the actual BOD_5 removal within filter beds reported in Table 7.3 represents only a part of total BOD_5 removal by a complete RBF system. Table 7.3 also includes BOD_5 removals with the recirculation tanks, which was determined by the projected BOD_5 into and the actual measured BOD_5 leaving the recirculation tank. Therefore, this chapter compared the actual measured BOD_5 removals with the theoretical removals at two different components, including i) BOD_5 removal within filter beds; and ii) BOD_5 removal in recirculation tanks.

Table 7.3 Summary of theoretical and actual measured organic matter removal rates for RBFs with different geotextile volume percentages

Geotexitle volume percentage	Theoretical total removal by RBFs	[1]Measured removal within filter beds		[2]Measured removal in recirculation tanks	
		Low HLR	High HLR	Low HLR	High HLR
0	90	63	62	26	8
25	87	77	74	64	7
50	83	42	4	61	29
75	82	57	30	49	12
100	78	29	38	63	28

[1]Measured removal within filter beds determined by the BOD_5 concentration measured in bench-scale system recirculation tank and the RBF effluent samples
[2]Measured removal in recirculation tanks was determined by the projected BOD_5 into and the actual measured BOD_5 leaving the recirculation tank.

BOD_5 Removal within the Filter Bed. The results presented in Table 7.3 show that the theoretical organic removal rate of the dual media RBFs decreased with increasing geotextile volume percentage. In particular, organic removal in the 100 % geotextile RBF was predicted to be much lower than the other RBFs based on Equation 7.3. However, the theoretical prediction does not mean that geotextile cannot remove organic matter as efficiently as crushed glass. In fact, the low organic removal rate from RBFs with a high geotextile volume percentage is caused by the lack of the depth of the geotextile layer, as Figure 7.19 indicates that minimum depth of geotextile layer to achieve organic matter removal greater than 90 % was 23 cm.

Table 7.3 also lists the actual measured BOD_5 removal rates by RBFs with different geotextile volume percentages. For the RBFs operated at the low HLR, the dual media RBF with the geotextile volume percentage of 25 % was found to provide the highest BOD_5 removal (e.g., 77 %). The 100 % crushed glass RBF was found to provide the second highest BOD_5 removal (e.g., 63 %) within the filter bed. Therefore, both the actual measured and the predicted BOD_5 removals within the filter bed found that RBFs with the geotextile volume percentage less than 25 % would provide higher BOD_5 removal than the other RBFs evaluated, although the predicted removal rates were higher than the measured removal rates. In addition, the measured results showed that the 100 % geotextile RBF provided the lowest BOD_5 removal (i.e., 29 %) within the filter bed at the low HLR among all tested RBFs in this chapter. This finding is also consistent with the theoretical prediction, that the 15 cm geotextile RBF would provide the lowest BOD_5 removal (i.e., 78 %) among all tested RBFs.

For RBFs operated at the high HLR, the dual media RBF with the geotextile volume percentage of 25 % was found to provide the highest BOD_5 removal (i.e., 74 %) within the filter bed, as shown in Table 7.3. The 100 % crushed glass RBF was found to provide the second highest BOD_5 removal (i.e., 62 %) within the filter bed. In addition, Table 7.3 shows that the 100 % geotextile RBF provided the lowest BOD_5 removal (i.e., 38 %) within the filter bed among all tested RBFs under the operation of the high HLR.

In summary, both the theoretical and the actual measured BOD_5 removals within filter beds showed the following observations:

- The geotextile RBF would provide the lowest BOD_5 removal among all tested RBFs in this chapter; and
- The dual media RBF with the geotextile volume percentage of 25 % could provide high BOD_5 removal within the filter bed.

BOD_5 Removal within the Recirculation Tank. Based on the conceptual model of air flow within the filter bed and the dissolved oxygen (DO) variation within the recirculation tank introduced in Chapter 5, more oxygen should be available in the recirculation tank of a RBF with a high geotextile volume percentage than that of a RBF with a low geotextile volume percentage, since geotextile has a higher porosity than crushed glass. Thus, more BOD_5 removal is expected in the recirculation tank of RBFs with a high geotextile volume percentage than that of RBFs with a low geotextile volume percentage, because more oxygen could be transferred into the recirculation tank of a RBF with a high geotextile volume percentage than with a low geotextile volume percentage. In particular, the conceptual model introduced in Chapter 5 indicates that the 100% crushed glass and the 100% geotextile RBFs would provide the lowest and the highest BOD_5 removals in the recirculation tank, respectively. Based on this conceptual air flow model, the degree of BOD_5 removal in the recirculation tank of the bench-scale RBF systems investigated in this chapter should follow the rank as shown in Figure 7.20.

Low

- 100 % Crushed Glass RBF
- 25 % Geotextile: 75 % Crushed Glass RBF
- 50 % Geotextile: 50 % Crushed Glass RBF
- 75 % Geotextile: 25 % Crushed Glass RBF
- 100 % Geotextile RBF

BOD_5 removal in the recirculation tank

High

Figure 7.20 Ranking of BOD_5 removal in the recirculation tank.

For the bench-scale RBFs dosed at the low HLR, Table 7.3 shows that the 100 % crushed glass RBF had the lowest BOD_5 removal (i.e., 26 %) in the recirculation tank among all tested RBFs in this chapter. Table 7.3 also shows that the 100 % geotextile RBF had a significantly higher BOD_5 removal (i.e., 63 %) in the recirculation tank. These observed ranking results are consistent with the conceptual air flow model prediction.

For the bench-scale RBFs dosed at the high HLR, Table 7.3 shows that the 100 % crushed glass and 25 % geotextile RBFs had the lowest BOD_5 removal (i.e., 8 and 7 %, respectively) in the recirculation tank as compared to the other RBFs evaluated. The 100 % geotextile RBF had a significantly higher BOD_5 removal (i.e., 28 %) in the recirculation tank. Again, these observed ranking results are also consistent with the conceptual air flow model prediction.

In summary, both the theoretical and the actual measured BOD_5 removals in recirculation tanks showed the following observations:

- The geotextile RBF would provide the highest BOD_5 removal among all tested RBFs in this chapter; and
- The crushed glass RBF would provide the lowest BOD_5 removal within the filter bed among all tested RBFs in this chapter.

Based on the analysis above and the data shown in Table 7.3, the dual media RBF with the geotextile volume percentage of 25 % could provide the most effective BOD_5 removal due to the following reasons:

i) The dual media RBF with the geotextile volume percentage of 25 % could theoretically provide high BOD_5 removal by biological reactions occurring both in the filter bed and the recirculation tank;
ii) The crushed glass RBF provided lower BOD_5 removal in the recirculation tank than the dual media RBF with the geotextile volume percentage of 25 %; and
iii) The 100 % geotextile RBF provided lower BOD_5 removal within the filter bed than the dual media RBF with the geotextile volume percentage of 25 %.

7.5 Conclusion

This study investigated a novel dual media RBF with crushed glass and geotextile for domestic wastewater treatment. The results showed that the volume ratio between crushed glass and geotextile of 3:1 (e.g., geotextile volume percentage of 25%) could provide better substrate removals (BOD_5, turbidity, and NH_4^+-N) than dual media RBFs with geotextile volume percentages of 50 and 75 %. More importantly, the dual media RBF with the geotextile volume percentage of 25 % which was operated at the HLR of 0.40 m^3/m^2/day could produce an effluent BOD_5 concentration lower than the 100 % crushed glass RBF operated at a lower HLR of 0.20 m^3/m^2/day. These results suggest that dual media RBF designs could potentially decrease the footprint requirements for filter bed significantly.

In addition, this study evaluated the performance of ten RBFs based on the water quality in the recirculation tank and after biofiltration. The results show that BOD_5, TN, and NH_4^+-N removal occurred in both the filter bed and the recirculation tank. Based on a simple model, this chapter predicted the maximum organic loading rates crushed glass and geotextile filter beds. The results of the model application indicates that high geotextile volume percentage could result in a low BOD_5 removal within the filter bed, since geotextile filter needs deeper media layer to achieve its maximum organic loading rate. However, considering improved capabilities to achieve higher BOD_5 removal within the recirculation tank of a RBF system with higher percentages of geotextile medium, this chapter concluded that the dual media RBF with geotextile volume percentage of 25 % could provide overall higher BOD_5 removal than both the crushed glass and the geotextile RBFs.

8. SUMMARY AND CONCLUSIONS

The overall objective of this research project was to develop an improved understanding of recirculating biofilters that would lead to a more robust design approach for the sustainable implementation of these onsite wastewater treatment systems. Based on the experimental results of controlled bench/pilot RBFs systems and a full-scale field treatment plant, the following sections presents a summary of the main research findings and outlines the conclusions of this work.

Alternative Filter Media

Although this study only compared four different filter media, the results present important research findings in terms of advancing this technology along sustainable development pathways. This study found that crushed glass and sand performed similarly as a biofilter media at both the bench- and field-scale. This conclusion indicates that crushed glass has the potential to replace sand as a biofilter media for geographic regions where sand is not readily available. This thesis also found that geotextile, a synthetic man-made material, could be another effective alternative filter medium to sand in RBF design. However, peat was found to be less effective as a filter medium for RBFs, even though it has been widely used in single-pass biofilter systems. Based on the results of the bench- and field-scale evaluations, analysis of effluent samples from the peat RBFs consistently demonstrated BOD_5 and NH_4^+-N concentrations higher than Canadian Water Quality Guidelines.

Practically, the following conclusions can be drawn:

- Crushed glass is an appropriate alternative to sand and should be considered more widely for RBFs for municipal and domestic wastewater treatment. Crushed glass is a readily available medium in developed countries where pollution prevention initiatives have promoted the recycling of certain manufacturing products. In addition, it is anticipated that the costs of crushed glass will be beneficial to most markets compared to sand if it is applied as the alternative filter medium; and

• Geotextile should be considered as an innovative medium for RBFs, especially where the system footprint is a concern. Even though this material presents a more expensive medium option to employ in full-scale operations, the results of this study found that geotextile RBFs could be dosed at high HLRs resulting in the reduction of overall design footprints and thus, operational costs.

Improved Understanding of Impact Factors on RBFs Performance

Previous successful working experience has suggested typical design criteria for RBFs, such as HLRs of 0.12-0.20 m^3/m^2/day, dosing frequencies of 24 to 200 times per day and recycle ratios of 2:1 to 6:1. However, current literature poorly describes the impact of these parameters on RBF performance. This study experimentally examined the impact of these factors on RBF performance through targeted bench-, pilot-, and field-scale investigations. The results of these studies found that dosing frequency significantly impacted RBF performance in terms of BOD_5 removal (α=0.05), as shown in Appendix H. Further investigation of dosing frequency found that 192 times per day was the optimal value to produce a minimum and stable effluent BOD_5 concentration among five different dosing frequencies, including 48, 96, 144, 192 and 240 times per day.

SEM images were taken to investigate the biofilm structure on the filter media over the entire filter bed depth. These images showed that biofilm was not distributed uniformly over the entire filter bed depth. Specifically, a less developed biofilm was observed at the bottom of the filter bed in comparison to biofilm development at the top of the filter bed. Therefore, the role of filter depth was also discussed in this thesis through the performance comparison of a 15-cm bench-scale and 30-cm pilot-scale RBF that were both operated with crushed glass as the filter medium. The results indicated that there was no significant difference in terms of the removal of investigated water quality parameters (e.g., BOD_5, turbidity, NH_4^+-N, and TN), as shown in Appendix D.

In addition, pilot-scale RBFs were used to evaluate the impact of three different ventilation locations on RBF performance, including at the top, the bottom sidewall, and both the top and the bottom sidewall of the filter beds. The results showed that BOD_5 removal could be improved by the RBF with ventilation at both the top and the bottom sidewall of the filter bed.

Practically, the following conclusions can be drawn from the results of this research:

- HLRs between 0.12 to 0.20 m^3/m^2/day did not impact RBF performance significantly (α=0.05). Therefore, it is possible that the footprint of RBF systems could be reduced by implementing high HLRs, such as 0.20 m^3/m^2/day instead of 0.12 m^3/m^2/day in RBF design;
- Recycle ratios between 2:1 and 4:1 did not impact RBF performance significantly (α=0.05). This research finding is of practical importance for regions where available construction area is a concern since it may be possible to reduce footprint requirements for the recirculating tank by decreasing recycle ratios;
- A high dosing frequency resulted in improved BOD_5 removal in five bench-scale RBFs.. This thesis found that a dosing frequency of 192 times per day was the optimal value as compared to the dosing frequencies of 48, 96, 144 and 240 times per day. This result would be important for regions where there are more stringent effluent discharge regulations for BOD_5 (i.e., inland surface water versus coastal discharge) in that adjustment of dosing frequency represents a simple and easily applied optimization strategy to achieve regulatory compliance;
- Ventilation location was found to be an impact factor for BOD_5 removal efficiency. The optimal location for filter bed ventilation was found to be from both the top and the bottom sidewall of filter beds. The experiments involved the introduction of

air by diffusion and concurrent flow, suggesting that extra power demands and footprint could be avoided.

Evaluation of Long-Term Field-Scale RBFs for Domestic Wastewater Treatment

This study evaluated the long-term performance of RBFs at the full-scale in terms of effluent water quality as well as the function of individual system components of RBFs. It was found both crushed glass and geotextile were effective alternative filter media to conventional sand designs. In addition, this thesis found that organic loading rates could be as high as 0.070 kg/m^2/d (for sand, geotextile or crushed glass RBFs). This results of this thesis also found that the recirculation tank was the key component for TN removal.

In conclusion, the field-study results of this thesis indicate the following practical considerations in the application of RBFs for septic tank effluent treatment:

- Crushed glass and geotextile could be effectively applied as RBF media following a long-term field-scale observation. Furthermore, this thesis found that peat was not an effective RBF medium based on bench- and full-scale experimental results that demonstrated poor BOD_5 and NH_4^+-N removals;
- RBFs could effectively treat wastewater with organic loading rates as high as 0.070 kg/m^2/day, which was approximately three times that recommended by the USEPA (2002). This conclusion can potentially broaden the application of RBFs; and
- The field-scale results of this thesis found that RBFs were not able to effectively remove TP. In addition, it was observed that measured system effluent fecal coliform counts were generally higher than the 200 CFU/ 100mL guideline recommended by CWQG. However, subsequent disinfection of biofiltration effluents (e.g., UV) prior to discharge is a commonly applied treatment strategy that would ensure RBF designs would be in compliance with local regulations.

Development of Novel Dual-Media RBFs

This study developed and investigated new dual media RBFs with crushed glass and geotextile for domestic wastewater treatment. The results showed that the dual media RBF with the geotextile volume percentage of 25 % could provide better substrate removals (BOD_5, TSS, and NH_4^+-N) than other dual media RBFs with the geotextile volume percentages of 50 and 75 %. More importantly, the results of this study demonstrated that operation of the dual media RBF with the geotextile volume percentage of 25 % at a high HLR of 0.40 $m^3/m^2/d$ could produce effluent BOD_5 concentrations lower than the crushed glass RBF operated at a low HLR of 0.20 $m^3/m^2/d$. This research result is important in that filter bed surface areas could potentially be reduced by 50 %, since HLRs and daily flow rated determine the footprints of the filter bed.

In conclusion, this phase of the research indicates that a dual media RBF with a geotextile volume percentage of 25 % could be successfully applied as an innovative biofiltration design, based on bench-scale evaluations. It is anticipated that this new design could potentially reduce the overall footprint requirements of RBFs by operating at substantially higher HLRs.

9. RECOMMENDATIONS

The purpose of this chapter is to present several key recommendations based on the results of this project to further advance the application of RBF technology for municipal and domestic wastewater treatment in rural and small communities. Specifically, recommendations for future research include:

9.1 RBFs Performance

Optimized TN Removal

One of the advantages of multiple pass biofilters over single-pass biofilter systems is that multiple pass biofilters can be more effective for TN removal than single pass biofilters (USEPA, 2002). This research project compared different impact factors (i.e., HLRs, recycle ratio, dosing frequency and filter media) on RBF performance as well as provided a novel dual-media design for enhanced BOD_5 removal with high HLRs. However, the results of this study found that the average TN removal varied widely from 20 to 70%. . Although the field-scale RBFs reduced TN by about 70 % as shown in Chapter 6, the effluent TN was found to be greater than 30 mg/ L. In some parts of North America, discharge regulations for effluent TN concentrations are less than 30 mg/ L. For example, the State of New Mexico, USA, requires that effluent nitrogen be less than 27 mg/ L. Therefore, additional research into enhanced TN removal in RBF systems should be initiated. The results presented in Chapter 7 demonstrated that the recirculation tank was the main system component responsible for TN removal. This finding suggests that enhanced TN removal could be potentially achieved through targeted investigations into the system dynamics of the recirculation tank.

Improved TP Removal

The results of this thesis showed that RBFs treating residential wastewater could not effectively remove TP. A review of literature shows that there are minimal number of technologies capable of achieving advanced phosphorus for onsite wastewater system applications. The technologies that have been successfully applied fall into the general categories of chemical, physical and biological systems. The controlled addition of chemicals such as aluminum, iron, and calcium based coagulants with subsequent flocculation and sedimentation has had only limited success because of inadequate operation, mechanical equipment problems and problems associated with excessive sludge production. Physical and chemical processes such as ion exchange and precipitation of phosphates have been evaluated with limited success (USEPA, 2002). Most notable successes have come with specialized filter materials that contain high concentrations of the above chemicals, but their service lives are finite. Studies of high iron sands and high aluminum mud indicate that 50 to 95 % of phosphorus can be removed (USEPA, 2002). However, the life of these systems has yet to be determined, after which the filter media will have to be removed and replaced. The use of supplemental iron powder mixed with natural sands is also being researched (USEPA, 2002). All calcareous sands and other sands with high concentrations of these three elements will exhibit high phosphorus removal rates for some finite periods. Typical calcium-containing US sands will essentially exhaust their capacity in 3 to 6 months, after which they will remove only particulate-based organic phosphorus or about 10 to 20 percent of the phosphorus contained in the wastewater.

The other known P-removal approach is the use of sequencing batch reactors (SBR) which can improve phosphorus removal in wastewater by proper sequencing of aeration periods (USEPA, 2002). Other aerobic biological units can result in improved phosphorous removal performance by the addition of anaerobic steps up to an effluent limit of 1 to 2 mg/ L, but the data to support the onsite applications of these types of technologies are lacking (USEPA, 2002).

Since engineering design that results in straightforward maintenance and operation of onsite wastewater treatment systems is critical for their application, more complicated technologies such as SBRs would not be practical for the individual property owner. Therefore, more studies should be conducted that evaluate TP removal in RBF systems for onsite wastewater treatment as this technology has the potential to presents a simple design solution for rural or small systems.

Possible Biofilter Clogging Problem

High organic loadings to RBF systems are a concern because of the associated clogging problems in the filter bed due to the biological processes associated with biodegradation. Avnimelech and Nevo (1964) revealed that a low C:N ratio suppressed clogging. Bouma (1979) and De Vries (1972) reported that biological clogging was due to the accumulation of suspended solids and biological growth under anaerobic conditions, which can result from high organic loadings since more oxygen will be consumed. Siegrist (1987) suggested that the main causes of clogging were elevated suspended solids and BOD_5 concentrations. The results of a nine-month field study of a full-scale RBF system treating residential wastewater found that BOD_5 loading rates could be as high as 0.070 kg/m^2/day maintaining high effluent quality without observed clogging problems. However, it has been reported that it could take years for clogging problems to emerge in RBF systems. Therefore, more long-term field-scale RBF studies need to be conducted, in order to establish a more robust understanding between organic loading rates and filter clogging occurrences.

9.2 RBFs Advanced Design Tool --- Mathematical Models

Mathematical models provide a powerful tool for understanding wastewater processes and describing the performance of RBFs as a function of design, operational, and environmental factors. Proper and careful use of mathematical biofiltration models can enable optimization of system design and operation, as well as provide a quantitative understanding of how design, operational and environmental factors impact pollutant treatment. Therefore, this thesis recommends a further study to focus on developing a mathematical model which integrates physical, chemical and biological reactions in the system, such that it could be used as a design tool for treatment prediction and system trouble shooting.

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APPENDIX A: FACTORIAL DESIGN ANALYSIS FOR BENCH-SCALE RSFs

Table A.1 Biofiltration – BOD_5 (dosing frequency of 96 times per day)

Original test

Date		Crushed glass				Sand				Raw WW
		HLR=0.12 m^3/m^2/day		HLR=0.20 m^3/m^2/day		HLR=0.12 m^3/m^2/day		HLR=0.20 m^3/m^2/day		
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	
6-Sep-03	Day 30*	3	9	2	2	3	7	2	2	161.5
7-Sep-03	Day 32	5	10	12	14	6	12	20	15	173
8-Sep-03	Day 33	1	6	6	4	2	7	5	6	155
9-Sep-03	Day 34	4	8	5	8	5	8	8	10	167
10-Sep-03	Day 35	9	5	30	30	10	10	18	16	189
11-Sep-03	Day 36	3	5	5	5	7	6	5	5	171

* start data point employed for system performance evaluation and comparison.

Table A.2 Duplication test

Date		Crushed glass				Sand				Raw WW
		HLR=0.12 m3/m2/day		HLR=0.40 m3/m2/day		HLR=0.12 m3/m2/day		HLR=0.40 m3/m2/day		
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	
20-Jan-04	Day 06	77	68	47	59	36	83	80	76	276
26-Jan-04	Day 12	67	81	83	85	66	64	76	82	173
29-Jan-04	Day 15	66	63	64	32	49	60	58	82	354
4-Feb-04	Day 21	23	12	27	33	31	26	38	31	373
7-Feb-04	Day 24	22	30	34	25	41	27	35	36	342
11-Feb-04	Day 28	15	35	50	42	44	21	30	21	382
16-Feb-04	Day 33*	19	32	63	42	30	14	42	18	162
21-Feb-04	Day 38	15	9	33	21	15	5	24	14	160
22-Feb-04	Day 39	8	10	11	28		4	5	6	150
24-Feb-04	Day 41	12	9	6	6	3	7	20	11	155
25-Feb-04	Day 42	11	7	26	29	22	2	25	14	180

* start data point employed for system performance evaluation and comparison.

Table A.3 Biofiltration – BOD_5 (dosing frequency of 48 times per day)
Original test

Date		Crushed glass				Sand				Raw WW
		HLR=0.12 m^3/m^2/day		HLR=0.20 m^3/m^2/day		HLR=0.12 m^3/m^2/day		HLR=0.20 m^3/m^2/day		
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	
3-Nov-03	Day 18*	3	6	4	3	6	3	5	7	100
6-Nov-03	Day 21	15	2	14	15	9	13	9	8	125
10-Nov-03	Day 25	9	6	7	5	4	4	6	7	107
13-Nov-03	Day 28	11	10	19	21	27	19	21	22	114
16-Nov-03	Day 31	11	12	10	19	8	20	17	8	128
18-Nov-03	Day 33	13	12	6	9	9	16	10	15	147
20-Nov-03	Day 35	34	29	32	27	24	22	29	33	179

* start data point employed for system performance evaluation and comparison.

Table A.4 Duplication test

Date		Crushed glass				Sand				Raw WW
		HLR=0.12 m3/m2/day		HLR=0.20 m3/m2/day		HLR=0.12 m3/m2/day		HLR=0.20 m3/m2/day		
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	
16-Mar-04	Day 18*	20	13	45	25	67	13	75	30	280
17-Mar-04	Day 19	37	25	23	53	41	18	34	21	280
23-Mar-04	Day 25	30	23	45	23	30	25	23	19	340
24-Mar-04	Day 26	23	37	17	16	27	20	20	27	390
25-Mar-04	Day 27	34	28	17	35	17	26	16	28	390
26-Mar-04	Day 28	25	20	16	25	19	27	20	20	390
30-Mar-04	Day 32	34	25	17	17	29	24	24	29	350
31-Mar-04	Day 33	33	33	32	33	35	30	32	36	350

* start data point employed for system performance evaluation and comparison.

Table A.5 Biofiltration – TN (dosing frequency of 96 times per day)

Original Test

Date		Crushed glass				Sand			
		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$	
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1
4-Sep-03	Day 28	4.8	6.8	16.8	19.6	32.0	20.0	6.0	28.0
5-Sep-03	Day 29	13.8	5.8	18.2	18.4	39.0	18.0	15.6	22.8
6-Sep-03	Day 30*	19.4	26.4	26.0	28.0	16.0	25.4	34.4	22.6
7-Sep-03	Day 32	21.2	33.0	25.2	31.4	20.4	18.0	29.6	17.6
8-Sep-03	Day 33	14.2	24.8	30.2	31.2	20.0	10.8	29.0	10.1
9-Sep-03	Day 34	14.2	20.8	33.4	29.2	4.4	17.8	28.8	6.8
10-Sep-03	Day 35	17.0	14.0	34.2	30.0	16.8	12.0	29.0	10.0
11-Sep-03	Day 36	8.4	15.4	4.6	28.0	6.0	16.4	27.8	32.0

* start data point employed for system performance evaluation and comparison.

Table A.6 Duplication test

Date		Crushed glass				Sand				Raw WW
		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$		
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	
27-Jan-04	Day 12	21.4	20.4	31.0	32.4	19.4	24.6	21.4	19.6	35.2
30-Jan-04	Day 15	23.6	20.4	23.2	22.6	21.2	21.8	24.4	15.4	34.8
4-Feb-04	Day 20	24.6	16.8	20.6	21.2	24.6	2.2	27.8	17.4	31.8
7-Feb-04	Day 23	25.8	15.8	24.4	19.0	33.6	12.2	31.0	16.4	48.2
8-Feb-04	Day 24	27.8	16.0	26.8	20.8	34.6	18.6	33.6	20.0	48.2
11-Feb-04	Day 27	26.8	8.2	17.8	19.4	32.8	18.4	17.2	9.2	47.6
16-Feb-04	Day 32*	25.0	6.2	23.2	20.4	46.1	24.6	17.2	3.6	55.0
18-Feb-04	Day 34	28.4	6.2	18.4	10.4	47.6	13.8	10.2	14.2	48.4
20-Feb-04	Day 36	19.2	4.0	11.4	12.4	40.8	4.6	7.2	5.0	53.8
22-Feb-04	Day 38	15.8	10.4	16.0	4.8	30.4	5.2	12.6	11.2	44.2
24-Feb-04	Day 40	13.6	20.4	18.6	5.8	28.0	5.0	9.5	13.6	44.0
25-Feb-04	Day 41	15.8	18.8	14.0	7.2	26.4	6.4	16.1	12.0	41.2

* start data point employed for system performance evaluation and comparison.

Table A.7 Biofiltration – TN (dosing frequency of 48 times per day)
Original test

Date		Crushed glass				Sand			
		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$	
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1
10-Nov-03	Day 25*	8.6	16.0	27.8	17.2	19.2	21.0	17.2	18.2
13-Nov-03	Day 28	6.6	3.8	9.0	5.2	10.0	10.0	9.0	3.2
16-Nov-03	Day 31	3.6	5.4	15.0	11.2	5.0	14.6	5.4	13.8
18-Nov-03	Day 33	3.4	5.4	15.6	9.4	3.8	9.4	4.6	10.0
20-Nov-03	Day 35	4.6	8.4	17.6	6.8	7.6	8.6	7.2	10.3

* start data point employed for system performance evaluation and comparison.

Table A.8 Duplication test

Date		Crushed glass				Sand				Raw WW
		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$		
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	
16-Mar-04	Day 7	11.4	31.4	32.0	15.6	36.4	6.8	29.6	17.6	49.8
18-Mar-04	Day 9	27.8	17.0	19.2	10.4	31.2	5.2	31.0	13.4	44.8
22-Mar-04	Day 13	10.2	13.6	21.4	9.0	20.4	4.2	13.2	8.2	47.8
23-Mar-04	Day 14	9.2	40.2	31.2	10.0	19.0	4.2	17.6	8.6	47.0
24-Mar-04	Day 15	13.8	14.4	31.8	10.8	13.6	5.4	17.4	10.1	47.0
30-Mar-04	Day 21*	17.8	7.8	6.0	6.0	15.8	6.8	16.2	14.0	52.4
31-Mar-04	Day 22	23.4	9.6	15.8	15.6	19.8	7.4	3.2	8.4	49.4
01-Apr-04	Day 23	32.2	27.8	40.0	29.8	17.8	11.0	25.6	13.0	55.0
02-Apr-04	Day 24	8.4	9.4	15.2	12.4	14.8	2.2	3.2	32.0	44.4
05-Apr-04	Day 27	7.8	14.2	20.8	24.2	26.4	8.8	7.4	25.4	50.4
07-Apr-04	Day 29	9.0	9.0	24.0	18.4	22.6	5.8	20.0	37.6	51.0

* start data point employed for system performance evaluation and comparison.

Table A.9 Biofiltration – NH_4^+-N (dosing frequency of 96 times per day)

Original test

Date		Crushed glass				Sand			
		HLR=0.12 m^3/m^2/day		HLR=0.20 m^3/m^2/day		HLR=0.12 m^3/m^2/day		HLR=0.20 m^3/m^2/day	
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1
3-Sep-03	Day 27	3.3	3.8	21.5	4.3	7.8	3.5	3.3	3.8
4-Sep-03	Day 28	4.8	1.0	17.0	1.8	3.3	1.0	5.5	1.5
5-Sep-03	Day 29		5.5	11.8	3.3	1.3	3.0	3.5	1.3
6-Sep-03	Day 30*	4.8	3.8	11.0	4.3	9.5	4.0	3.8	1.5
7-Sep-03	Day 32		1.3	7.8	2.3	3.0	0.5	0.0	2.5
8-Sep-03	Day 33	3.5	0.5	3.5	2.0	0.8	0.8	1.8	3.0
9-Sep-03	Day 34	0.0	0.3	1.3	1.8	0.5	0.3	2.0	
10-Sep-03	Day 35	4.3	0.3	2.5	3.0	0.5	0.0	0.8	0.5
11-Sep-03	Day 36	1.8	0.3	0.8	0.0	0.8	0.5	1.0	0.3

* start data point employed for system performance evaluation and comparison.

Table A.10 Duplication test

Date		Crushed glass				Sand			
		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$	
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1
21-Jan-04	Day 6	30.8	19.6	22.3	21.8	23.3	24.5	14.0	18.8
27-Jan-04	Day 12	7.0	16.6	32.6	28.6	20.8	0.8	7.1	4.0
30-Jan-04	Day 15	2.7	2.1	17.6	1.4	14.6	3.8	4.6	0.8
3-Feb-04	Day 19	3.3	6.4	25.8	7.8	16.5	0.8	11.0	13.5
4-Feb-04	Day 20	5.4	2.4	22.3	5.5	17.0	0.6	10.0	4.4
5-Feb-04	Day 21	5.4	1.8	22.0	5.8	15.8	0.9	6.0	2.4
7-Feb-04	Day 23	4.6	0.9	11.3	2.8	18.0	0.9	4.5	2.1
8-Feb-04	Day 24	3.8	0.8	9.4	2.5	16.3	0.6	2.8	2.1
11-Feb-04	Day 27	4.5	2.1	2.1	1.0	23.4	0.9	0.8	1.0
13-Feb-04	Day 29	3.9	1.1	5.5	3.0	17.5	1.3	1.9	2.6
16-Feb-04	Day 32*	1.5	2.1	6.4	2.3	11.1	2.4	1.0	1.5
18-Feb-04	Day 34	1.9		4.9	1.1	3.1		1.4	1.3
20-Feb-04	Day 36		1.5	2.0	1.3	0.9	1.0	1.0	1.4
22-Feb-04	Day 38	1.0	3.0	4.1	3.5		0.9	0.6	0.8
24-Feb-04	Day 40	1.1	1.6	2.9	0.1	0.1	0.1		0.1
25-Feb-04	Day 41	0.9	1.9	2.1	0.6	0.6	0.4	1.0	1.0

* start data point employed for systems performance evaluation and comparison.

Table A.11 Biofiltration – NH_4^+-N (dosing frequency of 48 times per day)
Original Test

Date		Crushed glass				Sand			
		HLR=0.12 m^3/m^2/day		HLR=0.20 m^3/m^2/day		HLR=0.12 m^3/m^2/day		HLR=0.20 m^3/m^2/day	
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1
28-Oct-03	Day 12	4.8	12.5	8.0	5.0	1.5	3.0	7.3	1.5
30-Oct-03	Day 14	1.0	0.8	4.5	0.5	1.5	1.3	5.0	0.3
10-Nov-03	Day 25*	0.8	0.5	0.5		0.3		0.3	0.5
11-Nov-03	Day 26	0.5	0.3	1.0	4.5	0.0	0.5	0.5	3.3
13-Nov-03	Day 28	0.5	0.3	1.0	0.3	0.5	1.5	0.0	0.3
17-Nov-03	Day 32	0.5	2.0	0.3	0.5	0.3	0.5	0.3	
18-Nov-03	Day 33	0.3		0.3	0.3	0.3	0.3	0.3	
20-Nov-03	Day 35	0.3	0.3	2.0	0.8	0.3	0.0	0.5	0.3

* start data point employed for systems performance evaluation and comparison.

Table A.12 Duplication Test

Date		Crushed glass				Sand				Raw WW
		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$		HLR=0.12 $m^3/m^2/day$		HLR=0.20 $m^3/m^2/day$		
		RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	RR=2:1	RR=4:1	
12-Mar-04	Day 03	27.9	26.0	32.9	32.5		11.3	31.5	12.4	
15-Mar-04	Day 06	0.6	21.0	17.9	2.8	20.1	2.0	29.0	3.1	
16-Mar-04	Day 07	0.9	18.1	17.1	8.0	21.5	2.0	23.4	2.1	
17-Mar-04	Day 08	2.3	5.8	6.0	4.3	22.4	2.1	19.0	0.8	42.8
18-Mar-04	Day 09	1.0	0.8	1.0	1.4	21.9	0.4	22.4	0.9	45.0
21-Mar-04	Day 12	0.9	0.4		0.5	7.1	0.1	2.4	0.8	
22-Mar-04	Day 13	0.5	1.4	0.6	0.6	3.9	0.3	2.8	0.9	43.5
23-Mar-04	Day 14	0.8	0.6	5.6	0.6	2.6	0.5	0.6	1.0	43.8
24-Mar-04	Day 15	0.8	1.6	1.0	0.9	0.9	0.5	0.8	1.0	44.3
25-Mar-04	Day 16	1.0	1.0	3.1	1.5	1.3	1.0	1.4	1.6	49.3
30-Mar-04	Day 21*	1.8	1.1	1.1	2.1	1.9	0.8	0.9	1.0	50.0
31-Mar-04	Day 22	1.6	1.1	0.8	1.6	0.9	0.8	0.9	1.3	49.5
2-Apr-04	Day 23	0.6	1.0	0.9	2.4	1.0	0.5	0.8	2.8	49.3
4-Apr-04	Day 25	0.8	1.1	1.4	1.5	0.8	0.8	0.8	2.0	49.0
7-Apr-04	Day 27	0.4	0.9	1.3	2.3	0.4	0.4	4.1	6.1	46.5

* start data point employed for system performance evaluation and comparison.

APPENDIX B: DOSING FREQUENCY

Table B.1. Biofiltration Effluents– BOD_5

Date		Dosing frequency (times per day)					
		48	**96**	**144**	**192**	**240**	**Raw**
21-Sep-04	Day 1	39	60			4	160
23-Sep-04	Day 3	8	40				160
24-Sep-04	Day 4	18	33	83	28	20	143
26-Sep-04	Day 6	24	53	4	24	36	160
27-Sep-04	Day 7	19	22			6	156
29-Sep-04	Day 9	21	22	17	12	9	150
30-Sep-04	Day 10		45	3	2	5	180
2-Oct-04	Day 12	32	35	26	32	36	180
3-Oct-04	Day 13	29	27	19	21	27	200
4-Oct-04	Day 14		27	22	19	21	226
5-Oct-04	Day 15	26	20				164
7-Oct-04	Day 17		19				164
9-Oct-04	Day 19		15				164
10-Oct-04	Day 20	23	20	10	7	16	220
11-Oct-04	Day 21*		14				132
12-Oct-04	Day 22	13	15	2		11	160
13-Oct-04	Day 23	7	3	4	5	3	160
14-Oct-04	Day 24		6				160
15-Oct-04	Day 25	18	16				160
16-Oct-04	Day 26		10				170
17-Oct-04	Day 27		9		3	5	164
19-Oct-04	Day 28	5	13		6	2	173
20-Oct-04	Day 29	30	7	14	6	2	164
21-Oct-04	Day 30	24	11	11	5	5	164

* start data point employed for system performance evaluation and comparison.

Table B.2 Biofiltration Effluents – NH_4^+-N

Date		Dosing Frequency (times per day)					Raw
		48	96	144	192	240	
21-Sep-04	Day 1	0.9	0.6			0.4	83.0
23-Sep-04	Day 3	0.6	0.5			0.5	87.0
24-Sep-04	Day 4	0.6	3.9	39.3	49.0	20.3	85.0
26-Sep-04	Day 6	1.0	1.3	11.1	12.3	20.6	101.0
27-Sep-04	Day 7	5.0	4.8			5.1	73.0
29-Sep-04	Day 9	1.8	1.4	1.5	14.9	1.1	70.5
30-Sep-04	Day 10	1.3	1.3	1.4	26.3	0.9	53.0
2-Oct-04	Day 12						
3-Oct-04	Day 13						
10-Oct-04	Day 20	1.5	1.6	0.6	0.6	0.9	58.5
12-Oct-04	Day 22*	1.1	1.1	0.9	0.1	0.3	17.5
13-Oct-04	Day 23	0.6	1.1	0.6		0.5	23.5
17-Oct-04	Day 27	0.5	0.6	0.5	0.6	0.6	
19-Oct-04	Day 28	0.6	0.6	0.6	0.3	0.1	30.6
20-Oct-04	Day 29	0.8	0.8	1.0	1.0	0.8	26.5
21-Oct-04	Day 30	0.6	0.9	0.9	0.6	0.6	28.3
24-Oct-04	Day 33	0.8	0.8	0.8	0.4	0.6	29.5

* start data point employed for system performance evaluation and comparison.

Table B.3 Biofiltration Effluents – TN

Date		Dosing Frequency (timer per day)					Raw
		48	**96**	**144**	**192**	**240**	
21-Sep-04	Day 1	2.5	17.0			2.1	110.0
23-Sep-04	Day 3	12.2	14.2			9.2	110.0
24-Sep-04	Day 4	4.6	32.8	55.0	55.0	35.0	110.0
26-Sep-04	Day 6	6.6	44.6	43.0	46.8	35.4	97.2
27-Sep-04	Day 7	5.0	44.8			5.1	73.0
10-Oct-04	Day 20	19.4	15.0	17.4	22.8	23.6	74.8
12-Oct-04	Day 22*	13.6		13.0	14.4	20.6	56.2
13-Oct-04	Day 23	16.4	13.8		17.2	14.2	53.2
17-Oct-04	Day 27	13.2		19.4	23.4	21.6	
19-Oct-04	Day 28	12.2	12.4	15.8	21.2	20.6	28.0
20-Oct-04	Day 29	12.2	13.8	21.8	23.0	15.4	32.0
21-Oct-04	Day 30	12.6	13.0	12.4	11.8	12.4	36.0

* start data point employed for system performance evaluation and comparison.

Table B.4 Biofiltration Effluents – Turbidity

Date		Dosing Frequency (times per day)				
		48	**96**	**144**	**192**	**240**
21-Sep-04	Day 1	2.9	1.9			1.7
23-Sep-04	Day 3	1.7	1.3			
24-Sep-04	Day 4	1.0		7.0	4.8	4.2
26-Sep-04	Day 6	1.6	3.7	5.4		2.9
27-Sep-04	Day 7	1.5	3.6			3.5
29-Sep-04	Day 9	13.8	4.5		2.0	2.5
30-Sep-04	Day 10					
02-Oct-04	Day 12*	2.0	3.0	1.6	1.4	4.7
03-Oct-04	Day 13	1.8	2.7	3.8	4.3	3.7
10-Oct-04	Day 20	3.0	2.3	8.7	2.4	1.3
12-Oct-04	Day 22	3.0		2.9		1.9
13-Oct-04	Day 23	2.2	4.3	5.7		1.4
17-Oct-04	Day 27	2.4	1.0		6.2	1.2
19-Oct-04	Day 28	2.3	1.3		5.6	1.4
20-Oct-04	Day 29	1.9	1.2	3.8		1.0
21-Oct-04	Day 30	1.2	1.3	4.4		1.2
24-Oct-04	Day 33	2.4	2.2	4.3		1.6

* start data point employed for system performance evaluation and comparison.

APPENDIX C: FOUR TYPES OF BENCH-SCALE RBFS

Table C.1 Biofiltration Effluents – BOD_5

Sample	Sand	Crushed glass	Peat	Geotextile
1	11	19	21	2
2	14	8	9	9
3	6	14	11	10
4	5	14	16	14
5	4	17	11	3
6	7	15	13	3
7	2	6	16	4
8	7	17	4	2
9		2		

Table C.2 Biofiltration Effluents – TN

Sample	Sand	Crushed glass	Geotextile	Peat
1	18.4	32.8	14.0	17.6
2	24.6	3.4	2.0	15.6
3	13.8	10.4	2.8	2.6
4	4.6	1.4	1.6	4.2
5	5.2			
6	5.0			
7	6.4			

Table C.3 Biofiltration Effluents – NH_4^+- N

Sample	Sand	Crushed glass	Geotextile	Peat
1	0.4	4.5	3.3	8.5
2	2.4	5.8	2.5	8.0
3	1.0	1.0	1.5	7.5
4	0.9	0.3	4.0	10.5
5	0.1	1.0	0.3	6.5
6	0.4	1.3	1.5	7.0
7		0.5	0.8	6.8
8		1.0	0.5	7.3
9			0.8	3.8
10			0.3	1.5

Table C.4 Biofiltration Effluents – pH

Sample	Sand	Crushed glass	Geotextile	Peat
1	7.8	7.4	6.6	4.4
2	7.9	7.8	5.9	3.6
3	7.8	7.0	5.1	3.3
4	7.9	7.5	5.4	3.4
5	8.0	7.3	6.4	3.5
6	8.0	7.3	6.6	3.7
7	7.6	7.4	7.3	4.1
8		7.3	7.1	4.0
9		7.7	7.4	4.2
10		7.2	7.6	4.1
11		7.7	7.6	4.8
12		7.6	7.7	4.8

Table C.5 Biofiltration Effluents – TP

Sample	Crushed glass	Geotextile	Peat	Raw Water
1	6.8	5.0	3.7	6.5
2	8.4	3.8	10.2	14.4
3	8.3	0.7	3.0	4.2
4	3.3	1.2	2.5	4.5
5	2.4	1.2	1.8	3.9
6	2.9	1.4	1.3	4.3

APPENDIX D: 15CM BENCH- AND 30CM PILOT-SCALE CRUSHED GLASS RBFS

Table D.1 Biofiltration Effluents – BOD_5

days	15cm Bench-scale RBF	30cm Pilot-scale RBF	Raw
1	60	15	90
3	40	27	90
4	33	21	143
6	53	31	107
7	22	17	156
9	22	21	150
12	35	22	325
13	27	19	226
20*	20	13	331
22	15	11	132
23	3	5	99
27	10	15	180
28	13	10	120
29	7	6	113
30	11	13	160

* start data point employed for system performance evaluation and comparison.

Table D.2 Biofiltration Effluents – NH_4^+-N

days	15cm Bench-scale RBF	30cm Pilot-scale RBF	Raw
1	0.6	0.6	83.0
3	0.5	4.1	87.0
4	3.9	0.8	85.0
6	1.3	5.1	101.0
7	4.8	5.3	73.0
9	1.4	1.4	70.5
10	1.3	1.1	53.0
20*	1.6	1.4	58.5
22	1.1	0.9	17.5
23	1.1	1.1	23.5
27	0.6	0.9	
28	0.6	1.6	30.6
29	0.8	1.3	26.5
30	0.9	0.6	28.3
33	0.8	0.6	29.5

* start data point employed for system performance evaluation and comparison.

Table D.3 Biofiltration Effluents – Turbidity

Date	days	15cm Bench-scale RBF	30cm Pilot-scale RBF
21-Sep	1	1.9	1.3
23-Sep	3	1.3	0.9
26-Sep	6	3.7	1.2
27-Sep	7	3.6	1.6
29-Sep	9	4.5	2.1
2-Oct	12	3.0	1.8
3-Oct	13	2.7	2.2
10-Oct	20*	2.3	2.6
13-Oct	23	4.3	2.9
17-Oct	27	1.0	2.6
19-Oct	28	1.3	3.2
20-Oct	29	1.2	2.0
21-Oct	30	1.3	1.7
24-Oct	31	2.2	3.5

* start data point employed for system performance evaluation and comparison.

Table D.4 Biofiltration Effluents – TN

days	15cm Bench-scale RBF	30cm Pilot-scale RBF	Raw
1	17.0	16.0	110.0
3	14.2	18.8	110.0
4	32.8	50.2	110.0
6	44.6	49.4	97.2
7	44.8	35.3	73.0
20*	15.0	13.4	74.8
23	13.8	15.6	53.2
28	12.4	16.2	28.0
29	13.8	17.2	32.0
30	13.0	12.4	36.0

* start data point employed for system performance evaluation and comparison.

APPENDIX E: VENTILATION LOCATIONS FOR PILOT-SCALE RBFS

Table E.1 Biofiltration Effluents – BOD_5

Date		Oxygen supply location			Raw
		Top	Bottom	Top & Bottom	
19-Jan-05	Day 01	30	41	13	153
22-Jan-05	Day 04	23	18	18	150
24-Jan-05	Day 06	47	56	4	123
26-Jan-05	Day 08	30	51	18	194
28-Jan-05	Day 10	6	9	15	190
31-Jan-05	Day 13	8	9	24	190
02-Feb-05	Day 15	36	10	26	177
04-Feb-05	Day 17	14	6	2	153
07-Feb-05	Day 20*	14	12		108
08-Feb-05	Day 21	19			195
09-Feb-05	Day 22	16			190
10-Feb-05	Day 23	10	16	2	236
11-Feb-05	Day 24	3	14	3	185
12-Feb-05	Day 25	13			110
13-Feb-05	Day 26	11			100
14-Feb-05	Day 27	2		2	154
15-Feb-05	Day 28	11		2	115
16-Feb-05	Day 29	7	19	2	137
17-Feb-05	Day 30	10	33	2	142
18-Feb-05	Day 31		24		213

* start data point employed for systems performance evaluation and comparison.

Table E.2 Biofiltration Effluents – Turbidity

Date		Oxygen supply location		
		Top	Bottom	Top & Bottom
19-Jan-05	Day 01	2.8	22.1	1.6
22-Jan-05	Day 04	1.6	4.0	1.0
24-Jan-05	Day 06	1.6	5.9	1.1
26-Jan-05	Day 08	2.4	1.2	1.1
28-Jan-05	Day 10	1.0	0.9	1.3
31-Jan-05	Day 13	0.6	0.7	1.1
02-Feb-05	Day 15	0.9	0.9	
04-Feb-05	Day 17	0.7	1.9	
07-Feb-05	Day 20*	1.5	1.2	
08-Feb-05	Day 21	2.6		1.6
09-Feb-05	Day 22			3.5
10-Feb-05	Day 23	1.0	6.6	1.5
11-Feb-05	Day 24	0.9	3.6	3.6
14-Feb-05	Day 27	3.9	3.6	5.0
15-Feb-05	Day 28	1.0		2.5
16-Feb-05	Day 29	0.9	3.9	2.1
17-Feb-05	Day 30	1.0	1.4	1.6
18-Feb-05	Day 31	0.9	1.0	2.9

* start data point employed for systems performance evaluation and comparison.

Table E.3 Biofiltration Effluents $-NH_4^+$-N

Date		Oxygen supply location			Raw
		Top	Bottom	Top & Bottom	
19-Jan-05	Day 01	23.3	30.5	1.3	37.0
22-Jan-05	Day 04	9.0	22.3	0.9	35.0
24-Jan-05	Day 06	11.6	13.3	0.5	54.5
26-Jan-05	Day 08	1.0	9.6	5.0	44.0
28-Jan-05	Day 10	1.3	2.4	5.4	51.3
31-Jan-05	Day 13	1.1	0.8	1.1	30.8
02-Feb-05	Day 15	0.8	1.3	1.1	34.3
04-Feb-05	Day 17	0.0	0.0	2.1	31.1
07-Feb-05	Day 20*	0.8	0.9		30.4
08-Feb-05	Day 21	0.9	0.0		33.0
09-Feb-05	Day 22	1.0	1.0		37.8
10-Feb-05	Day 23			1.8	
11-Feb-05	Day 24	0.1	0.0	0.6	31.8
14-Feb-05	Day 27	0.0	0.0	2.0	33.0
15-Feb-05	Day 28	0.0		1.0	39.8
16-Feb-05	Day 29	0.0	1.8	0.4	30.9
17-Feb-05	Day 30	0.0	0.0	0.6	34.0
18-Feb-05	Day 31	2.1	0.3		39.3

* start data point employed for systems performance evaluation and comparison.

Table E.4 – Biofiltration Effluents –TN

Date		Oxygen supply location			Raw
		Top	Bottom	Top & Bottom	
19-Jan-05	Day 01				
22-Jan-05	Day 04	42.0	42.4		54.4
24-Jan-05	Day 06	45.6	43.6	48.4	49.6
26-Jan-05	Day 08	14.4	27.6	43.6	23.2
28-Jan-05	Day 10	37.2	31.6	50.2	45.2
31-Jan-05	Day 13	36.0	31.6		35.2
02-Feb-05	Day 15	26.8	24.0		31.2
04-Feb-05	Day 17	20.0	18.8		29.6
07-Feb-05	Day 20*	27.6	27.6	32.0	33.6
08-Feb-05	Day 21	32.4		34.4	32.4
09-Feb-05	Day 22			32.0	30.8
10-Feb-05	Day 23	27.2	36.8	14.2	38.8
11-Feb-05	Day 24	22.8	33.6	14.8	44.8
14-Feb-05	Day 27	33.6	40.0	22.4	49.2
15-Feb-05	Day 28	20.0		23.2	28.8
16-Feb-05	Day 29	26.0	33.6		40.4
17-Feb-05	Day 30	18.8	16.0		32.8
18-Feb-05	Day 31	28.0	26.8		47.2

* start data point employed for systems performance evaluation and comparison.

Table E.5 – Biofiltration Effluents –Fecal coliform

Date		Oxygen supply location			Raw
		Top	Bottom	Top & Bottom	
19-Jan-05	Day 01			6.00E+07	1.00E+05
22-Jan-05	Day 04			5.00E+05	2.40E+05
24-Jan-05	Day 06			1.00E+06	3.00E+05
26-Jan-05	Day 08			1.00E+05	2.00E+05
28-Jan-05	Day 10	9.00E+02		1.00E+05	5.30E+05
31-Jan-05	Day 13	6.00E+01	4.60E+03		1.00E+05
02-Feb-05	Day 15	6.00E+02	1.40E+03		1.00E+05
04-Feb-05	Day 17	2.80E+03	1.00E+02	1.00E+04	1.00E+05
07-Feb-05	Day 20*				1.00E+05
08-Feb-05	Day 21			1.00E+03	1.00E+05
09-Feb-05	Day 22			6.20E+02	6.00E+05
10-Feb-05	Day 23	1.00E+01	1.30E+03	3.70E+02	1.00E+05
11-Feb-05	Day 24	4.30E+02	1.00E+01	1.00E+02	
14-Feb-05	Day 27	4.00E+02	6.00E+02	1.00E+02	
15-Feb-05	Day 28	1.00E+02	2.00E+02	1.00E+03	
16-Feb-05	Day 29	1.00E+02	1.00E+02	1.00E+02	
17-Feb-05	Day 30	1.00E+01	1.00E+01		

* start data point employed for systems performance evaluation and comparison.

Table E.6 Recirculation Tank Effluent – BOD_5

Ventilation at Top of the Filter Bed

Date	Measured recirculation tank effluent (mg/ L)	Measured filtration effluent (mg/ L)	Projected recirculation tank effluent (mg/ L)
07-Feb-05*	45	14	58
08-Feb-05	41	19	52
10-Feb-05	20	10	39
11-Feb-05	36	3	25
14-Feb-05	46	2	22
15-Feb-05	64	11	39
16-Feb-05	21	7	34
17-Feb-05	30	10	36

* start data point employed for systems performance evaluation and comparison.

Ventilation at Bottom Sidewall of the Filter Bed

Date	Measured recirculation tank effluent (mg/ L)	Measured filtration effluent (mg/ L)	Projected recirculation tank effluent (mg/ L)
07-Feb-05*	29	12	57
10-Feb-05	23	16	44
11-Feb-05	20	14	34
16-Feb-05	21	19	44
17-Feb-05	33	33	54
18-Feb-05	21	24	62

* start data point employed for systems performance evaluation and comparison.

Table E.7 Recirculation Tank Effluent – NH_4^+-N

Ventilation at Top of the Filter Bed

Date	Measured recirculation tank effluent (mg/ L)	Measured filtration effluent (mg/ L)	Projected recirculation tank effluent (mg/ L)
07-Feb-05*	5.3	0.8	6.7
08-Feb-05	8.5	0.9	7.3
09-Feb-05	10.5	1.0	8.4
11-Feb-05	8.4	0.1	6.5
14-Feb-05	5.1	0.0	6.6
16-Feb-05	6.1	0.0	6.2
17-Feb-05	6.6	0.0	6.8
18-Feb-05	12.1	2.1	9.6

* start data point employed for systems performance evaluation and comparison.

Ventilation at Bottom Sidewall of the Filter Bed

Date	Measured recirculation tank effluent (mg/ L)	Measured filtration effluent (mg/ L)	Projected recirculation tank effluent (mg/ L)
07-Feb-05*	3.1	0.9	6.8
08-Feb-05	3.0	0.0	6.6
09-Feb-05	5.0	1.0	8.4
11-Feb-05	2.8	0.0	6.4
14-Feb-05	1.0	0.0	6.6
16-Feb-05	4.5	1.8	7.6
17-Feb-05	6.3	0.0	6.8
18-Feb-05	10.6	0.3	8.1

* start data point employed for systems performance evaluation and comparison.

Table E.8 Recirculation Tank Effluent – TN

Ventilation at Top of the Filter Bed

Date	Measured recirculation tank effluent (mg/ L)	Measured filtration effluent (mg/ L)	Projected recirculation tank effluent (mg/ L)
07-Feb-05*	26.4	28.8	27.6
08-Feb-05	19.2	32.4	32.4
10-Feb-05	24.8	29.5	27.2
11-Feb-05	19.2	27.2	22.8
15-Feb-05	14.0	21.8	20.0
16-Feb-05	22.0	28.9	26.0
17-Feb-05	12.4	21.6	18.8
18-Feb-05	20.4	31.8	28.0

* start data point employed for systems performance evaluation and comparison.

Ventilation at Bottom Sidewall of the Filter Bed

Date	Measured recirculation tank effluent (mg/ L)	Measured filtration effluent (mg/ L)	Projected recirculation tank effluent (mg/ L)
07-Feb-05*	32.4	28.8	27.6
10-Feb-05	30.8	37.2	36.8
11-Feb-05	31.6	35.8	33.6
16-Feb-05	26.8	35.0	33.6
17-Feb-05	19.2	19.4	16.0
18-Feb-05	32.4	30.9	26.8

* start data point employed for systems performance evaluation and comparison.

Table E.9 Recirculation Tank Effluent – NO_3^--N

Ventilation at Top of the Filter Bed

Date	Measured recirculation tank effluent (mg/ L)	Measured filtration effluent (mg/ L)	Projected recirculation tank effluent (mg/ L)
07-Feb-05*	3.0	21.0	17.4
08-Feb-05	1.0	19.0	15.4
10-Feb-05	2.0	14.1	11.5
11-Feb-05	2.0	15.0	12.0
14-Feb-05	9.5	20.0	16.1
16-Feb-05	7.0	20.0	16.1
17-Feb-05	9.0	22.0	17.7
18-Feb-05	8.5	24.5	20.2

* start data point employed for systems performance evaluation and comparison.

Ventilation at Bottom Sidewall of the Filter Bed

Date	Measured recirculation tank effluent (mg/ L)	Measured filtration effluent (mg/ L)	Projected recirculation tank effluent (mg/ L)
07-Feb-05*	7.0	20.0	16.6
10-Feb-05	22.0	31.0	25.0
11-Feb-05	12.0	22.0	17.6
14-Feb-05	19.0	23.0	18.5
16-Feb-05	13.5	24.5	19.7
17-Feb-05	9.0	19.0	15.3
18-Feb-05	7.0	21.5	17.8

* start data point employed for systems performance evaluation and comparison.

APPENDIX F: FIELD SCALE RBFS

Table F.1 Septic Tank Effluent

Date	BOD	TN	NH_4^+-N	TP	TSS	Turbidity	pH	fecal coliform
07-Jun-04	276	90.0		8.4	30.0	58.3	8.4	
11-Jun-04	173	90.0		8.4	47.3	42.7	8.3	1.00E+06
15-Jun-04	344	90.0	56.8	6.9	85.0	47.9	7.9	6.00E+07
16-Jun-04	329	90.0	68.0	8.0	61.5	49.2	7.9	2.50E+06
17-Jun-04	307				17.6			6.00E+07
18-Jun-04								6.00E+07
22-Jun-04	468		74.0	8.6	20.0	61.5	8.3	2.00E+07
25-Jun-04	479	91.2	75.0		65.0		7.7	2.00E+07
12-Jul-04	379		84.8		37.5	50.4	8.3	6.00E+08
13-Jul-04	437	103.6	83.5	9.1	36.9	45.2	8.0	2.00E+08
14-Jul-04		110.0	102.0	10.1		47.1	7.9	4.40E+08
19-Jul-04	409	101.6	93.5	13.0		67.4	6.7	
20-Jul-04	374	110.0	124.5			53.7	6.8	
21-Jul-04	373	110.0				71.2	6.7	1.00E+06
22-Jul-04	392	110.0				59.0	6.7	1.00E+06
26-Jul-04	423	194.2			32.0	93.4	6.8	1.00E+06
03-Aug-04		176.0			40.0	55.4	6.6	1.00E+07
04-Aug-04	396				64.4	69.4	6.7	1.00E+07
05-Aug-04	415	265.0			66.7	64.0	6.7	1.00E+07
09-Aug-04	398	203.0	176.0			144.0	7.3	1.40E+06
10-Aug-04	383	209.0	133.6			54.9	7.3	3.40E+05
11-Aug-04	383	220.0	173.6			52.5	7.3	1.00E+05
12-Aug-04	366	240.0				32.4	6.7	
16-Aug-04	390	230.0				73.0	6.8	

Table F.2 Sand Filter Effluent

Date	BOD	TN	NH_4^+-N	TP	TSS	Turbidity	pH	fecal coliform
07-Jun-04	4	40.6		8.0		7.8	6.2	
11-Jun-04	2	50.8		7.1	7	1.0	6.0	1.00E+04
15-Jun-04		46.4	0.6	6.9		1.3	5.8	3.00E+05
16-Jun-04	23	50.6	0.4	7.1	15	1.1	5.8	1.00E+05
17-Jun-04	9				11			9.00E+04
18-Jun-04								1.00E+05
22-Jun-04		55.0	0.6	9.2	117	4.1	5.7	1.00E+04
25-Jun-04			0.5		50		5.9	1.00E+04
12-Jul-04		44.2	0.8	5.7	53	7.0	6.3	1.00E+04
13-Jul-04		38.0	0.9	8.3	2	1.0	6.2	2.00E+05
14-Jul-04		38.2	1.4	8.0		4.5	6.2	1.00E+04
19-Jul-04	21	33.0	1.1	9.0			6.3	
20-Jul-04	25	37.5	0.0	8.3		2.2	6.1	
21-Jul-04	25	37.8		8.8		2.2	6.1	1.00E+05
22-Jul-04	15	38.4	0.1	8.6	9	4.3	6.2	5.00E+05
26-Jul-04	22		0.4	8.9	24	15.9	6.5	1.00E+05
03-Aug-04	4	27.4	1.3	9.7	47	19.6	6.5	2.00E+06
04-Aug-04	23	17.2	0.6	11.4	26	6.4	6.5	1.00E+05
05-Aug-04	22	18.6	1.4	11.5	5	3.4	6.5	6.00E+07

Table F.3 – Crushed Glass Filter Effluents

Date	BOD	TN	NH_4^+-N	TP	TSS	Turbidity	pH	fecal coliform
07-Jun-04	6	29.8		6.5	9	4.1	6.5	
11-Jun-04	2	45.8		7.7	6	0.8	5.8	1.00E+04
15-Jun-04	15	44.8	0.9	6.9		1.5	5.6	2.00E+04
16-Jun-04	22	38.8	0.1	6.9	2	1.1	5.9	1.00E+04
17-Jun-04	10				10			2.00E+05
18-Jun-04					23			2.00E+05
09-Aug-04	4	20.0	1.3	17.8		14.5	6.9	6.00E+03
10-Aug-04	6	24.0	1.6	20.8		2.9	7.1	5.00E+03
11-Aug-04	7	19.6	1.9	13.2		3.3	7.0	3.50E+04
12-Aug-04	15		1.0	17.0		1.1	6.5	
16-Aug-04	17		0.8	10.6		1.9	6.4	

Table F.4 – Peat Filter Effluents

Date	BOD	TN	NH_4^+-N	TP	TSS	Turbidity	pH	fecal coliform
22-Jun-04	18	48.2	20.1	7.9	11	2.4	5.9	1.00E+04
25-Jun-04	86		20.6		10		6.0	1.00E+04
12-Jul-04	4	45.2	15.3		28	4.9	6.3	
13-Jul-04	4	49.8	17.4	9.5	12	1.7	5.9	1.00E+04
14-Jul-04		48.2	17.6	8.8		2.0	5.8	1.00E+04
19-Jul-04	23	41.0	13.1	9.5		2.8	6.1	

Table F.5 Geotextile Filter Effluents

Date	BOD	TN	NH_4^+-N	TP	TSS	Turbidity	pH	fecal coliform
20-Jul-04	29	8.4	4.0	9.0			5.8	
21-Jul-04	5	39.6		9.4		4.6	5.9	1.00E+05
22-Jul-04	29	40.4	3.3	9.9		2.3	6.1	1.00E+05
26-Jul-04	8		3.3	9.1	16	5.1	6.1	4.00E+05
03-Aug-04	6	26.0	4.3		7	3.3	6.5	1.00E+05
04-Aug-04	15	25.8	4.1	11.6	18	2.2	6.5	2.00E+05
05-Aug-04	13	19.0	3.5	12.2	12	2.8	6.5	1.00E+05
09-Aug-04	6	24.8	4.9	18.2		8.0	6.9	1.10E+04
10-Aug-04	32	21.6	6.5	18.0		3.6	7.2	3.00E+06
11-Aug-04	26	24.4	5.3	13.8		2.5	7.2	1.00E+04
12-Aug-04	27		5.9	16.8		1.5	6.7	
16-Aug-04	30		4.5	10.4		5.4	6.6	

Table F.6 UV Effluents

Date	BOD	TN	NH_4^+-N	TP	TSS	pH	fecal coliform
07-Jun-04	2	38.8		6.7		6.7	
11-Jun-04	2	46.6		6.8	6	6.1	3.00E+04
15-Jun-04		49.4	3.9	6.0		5.8	3.50E+05
16-Jun-04		48.0	3.4		10	5.7	1.00E+04
17-Jun-04	9				13		1.00E+04
18-Jun-04					16		1.00E+04
22-Jun-04	44	50.8	3.6	8.6	5	5.9	1.00E+04
25-Jun-04	47		0.8		10	6.3	1.00E+04
12-Jul-04	10	45.8	4.6		2	6.2	1.00E+04
13-Jul-04	28	45.6	3.9	9.1	11	6.0	1.00E+04
14-Jul-04		42.4	2.3	10.1		5.9	1.00E+04
19-Jul-04	13	36.8	4.4	13.0		6.2	
20-Jul-04	25	37.8	4.3	8.3		6.0	
21-Jul-04	5	34.6		9.0		5.8	
22-Jul-04	20	40.6	1.8	10.1	5	6.2	
26-Jul-04	25		1.9		2	6.4	
03-Aug-04	6	26.4	3.0	11.0		6.5	
04-Aug-04	23	25.8	9.4	11.6	1	6.4	
05-Aug-04	27	22.2	1.3	11.8	2	6.5	
09-Aug-04	2	25.2	4.4	18.0		7.2	2.00E+03
10-Aug-04	9	20.8	3.4	13.0		7.2	4.00E+04
11-Aug-04	2	13.6	3.1	13.8		7.2	1.00E+03
12-Aug-04	21		2.9	21.8		6.5	
16-Aug-04	20		2.3	10.2		6.8	

Table F.7 UV Effluents

Date	BOD	TN	NH_4^+-N	TP	TSS	pH	fecal coliform
07-Jun-04	18	40.5		7.6	17	7.6	
11-Jun-04	14	51.8		7.4	28	6.4	
15-Jun-04	10	49.4	10	7.5		6.1	
16-Jun-04	46	45.2	5.5		18	5.9	
17-Jun-04	73				4		
18-Jun-04					60		
22-Jun-04	43	50.4	9.0	8.5	4	6.2	1.00E+04
25-Jun-04	76		9.3		7	6.2	2.00E+05
12-Jul-04		44.8	11.8		32	6.5	6.00E+07
13-Jul-04	35	42.8	12.4	9.1	15	6.3	2.00E+05
14-Jul-04		43.8	9.8	8.4		6.2	1.00E+04
19-Jul-04	50	37.2	16.4	9.9		6.4	
20-Jul-04	16	37.2	8.8	9.2		6.2	
21-Jul-04	11	42.2		8.8		6.2	
22-Jul-04	69	44.2	15.1	9.6	11	6.5	
26-Jul-04	35		13.6			6.4	
03-Aug-04	17	26.6	7.8	10.4		6.5	
04-Aug-04	23	24.6	2.1	11.9		6.5	
05-Aug-04	80	17.0	16.0	11.5		6.9	
09-Aug-04	17	34.0	10.3	17.8		7.1	5.00E+03
10-Aug-04	55	21.2	12.4	19.8		7.2	2.00E+04
11-Aug-04	23	13.2	10.9	13.8		7.1	1.30E+04
12-Aug-04	11		8.0	16.4		6.6	
16-Aug-04	9		12.0	11.6		6.6	

APPENDIX G: DUAL-MEDIA RBFS

Table G.1 Biofiltration Effluents BOD_5

Sample	CG		3CG:1GT		2CG:2GT		CG:3GT		GT		Raw
	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	
1	8	26	2	14	4	28	9	17	9	17	236
2	14	21	2	18	9	25	12	30	11	29	185
3	14	9	6	12	13	49	14	15	16	22	190
4	17	11	2	10	2	21	2	35	11	25	154
5	15	20	2	2	25	79	14	51	13	25	115
6	6	18	2	2	4	55	3	11	16		137
7	17	20		11		81		80	4		163
8	2	13		16		67		57			155
9		10		6				74			142
10		22						46			213
11		7									
12		17									

Table G.2 Biofiltration Effluents Turbidity

Sample	CG		3CG:1GT		2CG:2GT		CG:3GT		GT		Raw
	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	
1	1.7	2.6	1.6	1.4	0.9	7.9	8.3	1.9	1.1	11.4	72.0
2	1.4	2.0	0.9	1.3	1.5	7.6	5.3	2.6	1.3	11.3	77.6
3	1.6	2.8	1.3	1.2	2.1	8.4	5.0	2.6	1.4	2.8	57.1
4	1.5	2.8	1.3	1.4	2.5	5.6	2.3	3.5	0.9	22.7	62.4
5	1.4	2.4	1.1	1.0	1.2	15.4	5.1	5.7	0.8		45.5
6		2.5	1.3	0.7	2.8	8.1	1.9	2.1	0.9		44.0
7		3.9		0.8		8.3	0.0	3.6			
8		2.1		1.2		12.8	0.0	4.0			
9				0.9				5.0			
10				0.6				10.1			

Table G.3 Biofiltration Effluents NH_4^+-N

Sample	Crushed Glass		GT		3CG:1GT		2CG:2GT		1CG:3GT		Raw
	Low	High	Low	High	Low	High	Low	High	Low	High	
1	0.3	0.0	0.3	3.6	0.1	0.0	0.0	1.4	0.8	1.8	30.4
2	1.0	0.0	1.5	4.8	0.4	0.6	0.4	2.4	13.6	4.5	33.0
3	1.3	1.0	0.8	9.0	0.3	1.6	0.8	1.0	10.3	11.5	37.8
4	0.5	0.0	0.5	0.0	0.1	1.4	0.1	16.0	7.0	12.8	31.8
5	1.0	0.0	0.8	33.1	1.3	0.0	0.1	4.0	3.0	1.6	33.0
6	0.0	1.3	0.3	12.5	0.0	4.9	0.6	29.1	4.0	18.1	39.8
7		0.3			0.0	4.3	0.0	16.5	4.0	15.3	30.9
8		0.0				0.0		19.3		14.8	34.0
9		0.6				0.8		13.1		13.8	39.3
10											41.3

Table G.4 Biofiltration Effluents TN

Sample	Crushed Glass		GT		3CG:1GT		2CG:2GT		1CG:3GT		Raw
	Low	High	Low	High	Low	High	Low	High	Low	High	
1	32.8	2.4	14.0	12.4	1.2	13.2	2.8	3.6	12.8	0.0	32.4
2	3.4	8.4	2.0	13.2	12.4	12.4	13.0	6.4	23.4	4.0	30.8
3	10.4	14.8	2.8	36.8	30.0	16.4	32.0	23.6	33.6	4.8	38.8
4	1.4	8.0	1.6	27.6	11.6	34.8	12.0	44.8	13.8	14.0	44.8
5		20.8		20.4	19.2	25.6	5.4	34.4	11.0	38.8	49.2
6		18.0			12.8	19.2	8.2	40.4	14.6	26.4	40.4
7		3.2				6.4		30.4		32.0	32.8
8		26.8				25.6		37.2		24.8	47.2
9		17.2				14.0				32.0	40.2
10		18.8				18.4					44.5

Table G.5 Biofiltration Effluents NO_3^--N

Sample	CG		3CG:1GT		2CG:2GT		CG:3GT		GT		Raw
	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	
1	4.5	0.5	5.0	7.0	10.0	1.5	7.5	0.0	7.5	8.0	3.0
2	13.5	7.5	5.5	13.5	5.5	0.4	4.5	0.5	18.5	7.5	0.9
3	14.5	8.5	7.5	17.5	6.5	3.5	8.1	2.5	15.5	15.5	0.5
4	14.5	8.5	9.0	18.0	7.5	20.5	10.0	7.5	20.0	2.5	1.0
5	15.0	12.0	6.5	14.5	5.0	15.5	7.0	0.0	18.0	9.5	0.0
6	4.5	0.5	8.5	1.5	7.0	12.5	6.0	19.0	15.0		0.5
7		13.5		10.5		9.5		17.0			0.4
8		16.5		13.0		10.0		9.5			0.5
9		17.0		17.0				10.5			0.5
10		13.5		15.5				15.0			3.0

Table G.6 Biofiltration Effluents Fecal coliform

Sample	Crushed Glass		GT		3CG:1GT		2CG:2GT		1CG:3GT		Raw
	Low	High	Low	High	Low	High	Low	High	Low	High	
1	1.0E+02	4.5E+02	1.8E+03	6.0E+02	8.0E+01	1.0E+01	4.2E+02	5.0E+01	3.5E+02	3.0E+01	5.0E+05
2	1.0E+02	1.1E+02	2.0E+03	1.5E+03	6.0E+01	1.0E+01	2.0E+03	1.0E+02	1.5E+03	1.6E+02	6.0E+05
3	1.0E+03	3.0E+01	4.0E+03	3.3E+03	1.4E+02	1.0E+01	6.0E+02	6.0E+03	3.5E+02	1.1E+02	6.0E+05
4	1.0E+03	1.0E+01	1.0E+02	1.0E+03	2.0E+02	1.0E+01	6.0E+03	6.0E+03	1.0E+02	3.0E+01	2.0E+05
5	1.0E+03	1.0E+01	1.0E+02	1.0E+02	2.0E+02	1.0E+01	4.2E+03	6.5E+02	2.0E+02	4.2E+03	1.1E+06
6		5.0E+02			9.0E+02	1.0E+01	3.5E+03	6.0E+03	8.0E+02	1.0E+02	1.0E+06
7		7.0E+01				1.5E+02		6.0E+03		1.7E+02	
8		3.6E+02				1.0E+02				1.0E+01	
9		2.8E+03				1.0E+01					
10		3.5E+02									

Table G.7 Biofiltration Effluents pH

Sample	Crushed Glass		GT		3CG:1GT		2CG:2GT		1CG:3GT		Raw
	Low	High	Low	High	Low	High	Low	High	Low	High	
1	7.5	7.0	6.6	7.2	7.5	6.7	7.2	7.2	7.0	7.4	7.9
2	7.9	7.2	5.9	7.3	7.4	6.4	7.2	7.7	7.1	7.5	7.8
3		7.0	5.1	7.1	7.2	6.1	7.4	7.5	7.4	7.7	7.8
4	7.8	7.3	5.4	7.9	7.2	5.9	7.2	6.9	7.2	7.5	8.0
5	7.9	6.8	6.4	7.7		7.4		7.3		7.4	8.0
6	8.0	7.4	6.6			6.6		7.9		7.9	8.0
7	7.0	6.9	7.3			7.0		7.9		8.0	7.0
8	7.6	6.4	7.1			7.9		8.1		8.1	7.6
9		6.5	7.4			6.3		7.9		8.0	7.3
10		6.5	7.6								

Table G.8 Observed Recirculation Tank Effluent BOD_5

Sample	CG		3CG:1GT		2CG:2GT		CG:3GT		GT	
	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR
1	37	78	13	53	12	56	27	74	20	61
2		28	16	99	15	61	20	67	22	26
3	56	49	13	17	17	55	21	31	16	46
4	20	22	2	28	22	23	22	66	6	33
5	29		19	60	13	52	18	59	24	36
6	17	54	16	13	19	22	16		10	27
7		39		22		100				
8		27		37		40				
9		49		22		68				
10				39		52				

Table G.9 Observed Recirculation Tank Effluent NH_4^+-N

Sample	CG		3CG:1GT		2CG:2GT		CG:3GT		GT	
	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR
1	4.8	0.9	0.0	0.1	0.0	0.9	4.3	2.0	0.5	9.6
2	7.6	3.9	1.9	3.6	2.5	3.9	10.6	10.1	17.5	18.9
3	2.8	0.8	1.6	1.4	4.6	10.8	10.4	13.3	13.1	12.0
4	2.0	0.6	0.5	2.3	0.5	21.1	6.3	22.5	7.6	12.8
5	2.8	1.0	0.8	0.0	1.3	18.1	12.8	20.8	6.1	22.0
6	1.1	1.5	2.6	4.6	1.5	18.6	9.3	19.8	6.8	16.8
7	1.6	4.3	3.3	9.5	0.6	24.8		24.1	6.1	
8		1.1		0.0		25.9		15.6		
9		2.4		2.5		18.8		19.4		

Table G.10 Observed Recirculation Tank Effluent Turbidity

Sample	CG		3CG:1GT		2CG:2GT		CG:3GT		GT	
	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR
1	21.7	32.5	2.9	2.1	0.7	1.2	1.5	11.7	38.0	5.2
2	13.4	10.8	1.1	53.6	0.9	2.4	4.5	54.4	25.4	4.1
3	11.6	6.1	2.7	1.0	2.0	1.7	4.8	18.0	22.3	39.5
4	4.9	34.1	1.0	31.5	1.4	1.8	1.3	2.5	10.0	6.8
5	12.9	10.7	1.8	0.9	0.8	0.9	3.6	5.2	11.6	7.9
6		5.8		2.0	1.3	10.7		40.4		67.6
7		16.1		7.3		13.1		52.4		
8		4.8		5.2		5.6		10.8		
9		2.2		5.6		1.9		5.5		
10		12.2		0.7				1.1		

Table G.11 Observed Recirculation Tank Effluent TN

Sample	CG		3CG:1GT		2CG:2GT		CG:3GT		GT	
	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR
1	10.0	12.0	12.2	12.8	10.6	6.8	18.8	14.0	26.0	10.8
2	20.0	12.8	14.0	14.4	13.6	17.6	11.6	30.8	39.2	32.0
3	20.2	6.5	18.0	19.6	10.8		15.6	6.4	39.0	12.8
4	13.6	10.4	19.0	27.2	19.8	19.6	10.6		28.4	29.6
5	12.0	19.6	15.2	18.8	10.2		15.6		31.0	38.4
6	9.8	16.8		19.2		46.8		31.2		
7		11.2		14.4		55.2				
8		24.8		26.8		34.0		24.8		
9		14.8		12.8		26.0		32.8		
10		20.4		15.6		34.0				

Table G.12 Observed Recirculation Tank Effluent NO_3^--N

Sample	CG		3CG:1GT		2CG:2GT		CG:3GT		GT	
	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR
1	1.5	4.0	5	11.0	7.5	3.5	5.0	3.0	5.5	1.5
2	2.0	7.0	5.5	12.0	4.5		3.0		0.5	
3	6.0	8.0	7.0	6.5	4.5		5.0	2.5	2.0	4.0
4	5.5	8.5	6.5	6.5	7.5	0.4	9.0	2.0		9.5
5	3.5	11.0	6.5	15.0	4.0		5.0	0.5	1.0	1.5
6	2.5	0.5	0.8	1.0	5.0	4.0	4.0	3.5	1.5	9.0
7		13.0	3.5	1.8	2.0	2.5		4.5		
8		14.5		9.5	3.5	3.0		3.0		
9		6.0		6.0		2.5		3.5		
10		2.5		4.5		4.0		4.5		

Table G.13 Observed Recirculation Tank Effluent Fecal Coliform

Sample	CG		3CG:1GT		2CG:2GT		CG:3GT		GT	
	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR
1	3.25E+03	1.60E+04	5.00E+02	1.80E+04	3.00E+03	1.00E+03	3.00E+01	2.40E+04	6.45E+02	6.50E+03
2	3.50E+04	1.60E+04	1.80E+03	1.85E+03	2.00E+03	3.50E+03	2.00E+03	5.00E+03	5.35E+03	3.50E+03
3	4.90E+04	7.50E+03	7.00E+02	5.50E+03	6.40E+03	6.00E+03	1.40E+04	1.00E+03	1.80E+04	2.00E+04
4	7.00E+03	2.30E+04	3.00E+03	3.50E+03	2.00E+03	1.10E+04	2.00E+02	2.90E+04	6.00E+03	2.40E+04
5	6.00E+03	2.00E+03	5.00E+02	4.00E+02	4.30E+03	4.40E+03	2.70E+03	1.21E+04	6.00E+03	
6	6.00E+03	4.10E+03	1.60E+03	2.05E+03	6.00E+03	3.00E+03	1.80E+03	2.91E+04		
7		8.30E+03		5.00E+03		1.00E+03		2.20E+03		
8		1.60E+04		4.00E+03				2.00E+02		
9		3.20E+03		1.00E+03						

Table G.14 Observed Recirculation Tank Effluent pH

Sample	CG		3CG:1GT		2CG:2GT		CG:3GT		GT	
	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR
1	7.6	7.0	7.4	7.0	6.9	7.1	6.9	7.3	7.3	7.6
2	7.4	7.1	7.3	7.1	7.1	7.5	7.6	7.9	7.6	7.5
3	7.8	7.0	7.2	6.8	7.4	7.5	7.5	7.7	7.9	7.8
4	7.4	7.2	7.2	6.7	7.1	7.9	7.5	7.8	7.9	8.1
5		6.7		7.5		7.6		8.0		7.8
6		7.4		7.2		7.9		7.9		
7		7.0		7.4		8.1		8.1		
8		6.9		7.6		8.0		8.2		
9		6.8		6.8						
10		6.8								

APPENDIX H: FACTORIAL ANALYSIS

Table H.1

Design:	low	high
HLR (H)	3	5
Filter media (M)	Crushed glass	Sand
Dose frequency (D)	48 times/day	96 times/day
Recycle ratio(R)	2 to 1	4 to 1

exp. order	**effect**	H	**M**	**D**	**R**
1	-	-	-	-	-
2	H	+	-	-	-
3	M	-	+	-	-
4	HM	+	+	-	-
5	D	-	-	+	-
6	HD	+	-	+	-
7	MD	-	+	+	-
8	HMD	+	+	+	-
9	R	-	-	-	+
10	HR	+	-	-	+
11	MR	-	+	-	+
12	HMR	+	+	-	+
13	DR	-	-	+	+
14	HDR	+	-	+	+
15	MDR	-	+	+	+
16	HMDR	+	+	+	+

Yi (Removal)	Yate algorithm 1	2	3	4	Divisor	parameter estimate	Residuals
89	179.0	358.0	742.0	1481.0	16	92.5625	-3.6
90	179.0	384.0	739.0	-11.0	8	-1.375	-2.6
90	192.0	359.0	-6.0	-3.0	8	-0.375	-2.6
89	192.0	380.0	-5.0	5.0	8	0.625	-3.6
98	181.0	0.0	0.0	47.0	8	5.875	5.4
94	178.0	-6.0	-3.0	-5.0	8	-0.625	1.4
97	190.0	-3.0	0.0	3.0	8	0.375	4.4
95	190.0	-2.0	5.0	3.0	8	0.375	2.4
92	1.0	0.0	26.0	-3.0	8	-0.375	-0.6
89	-1.0	0.0	21.0	1.0	8	0.125	-3.6
89	-4.0	-3.0	-6.0	-3.0	8	-0.375	-3.6
89	-2.0	0.0	1.0	5.0	8	0.625	-3.6
96	-3.0	-2.0	0.0	-5.0	8	-0.625	3.4
94	0.0	2.0	3.0	7.0	8	0.875	1.4
95	-2.0	3.0	4.0	3.0	8	0.375	2.4
95	0.0	2.0	-1.0	-5.0	8	-0.625	2.4

Residual Plot

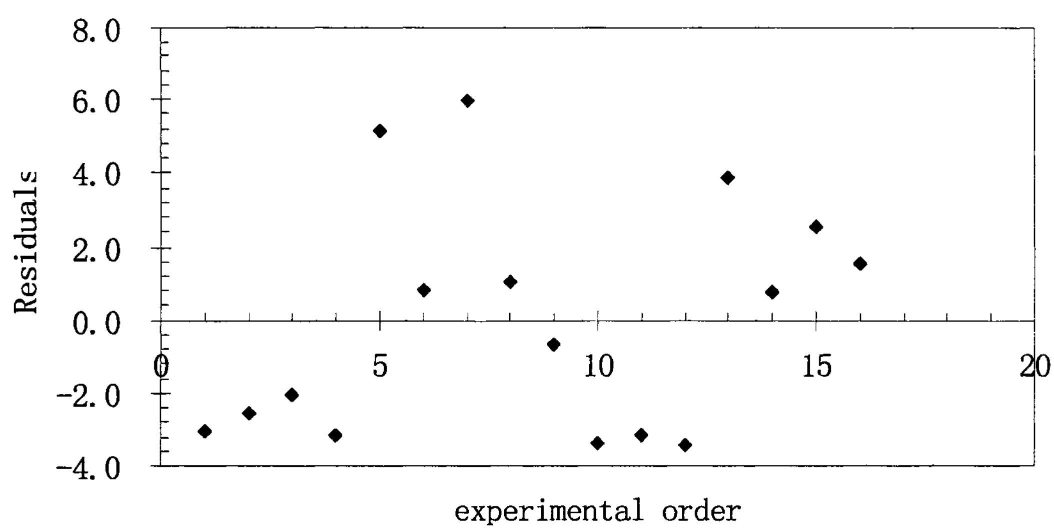
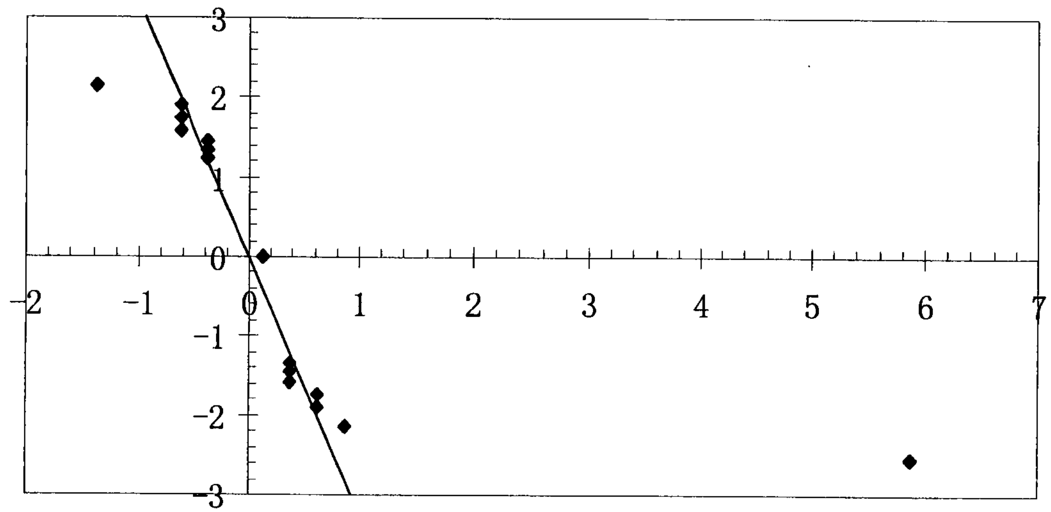


Table H.2 Blom Plot

factors		rank	$\frac{i-0.375}{m+0.25}$	v	sgn (v)	y	zp	Mean square
D	5.875	1	0.038462	-0.46154	-1	2.552684	-2.55268	2.157227
HDR	0.875	2	0.1	-0.4	-1	2.145966	-2.14597	0.047852
HM	0.625	3	0.161538	-0.33846	-1	1.909456	-1.90946	0.024414
HMR	0.625	4	0.223077	-0.27692	-1	1.732189	-1.73219	0.024414
MD	0.375	5	0.284615	-0.21538	-1	1.585318	-1.58532	0.008789
HMD	0.375	6	0.346154	-0.15385	-1	1.456621	-1.45662	0.008789
MDR	0.375	7	0.407692	-0.09231	-1	1.339584	-1.33958	0.008789
HR	0.125	8	0.469231	-0.03077	0	1.17741	0	0.000977
M	-0.375	9	0.530769	0.030769	1	1.230171	1.230171	0.008789
R	-0.375	10	0.592308	0.092308	1	1.339584	1.339584	0.008789
MR	-0.375	11	0.653846	0.153846	1	1.456621	1.456621	0.008789
HD	-0.625	12	0.715385	0.215385	1	1.585318	1.585318	0.024414
DR	-0.625	13	0.776923	0.276923	1	1.732189	1.732189	0.024414
HMDR	-0.625	14	0.838462	0.338462	1	1.909456	1.909456	0.024414
H	-1.375	15	0.9	0.4	1	2.145966	2.145966	0.118164



ANOVA

SOURCE	mean square	degree	F	$F_{(1,1,0.05)}$
	2.157226563	1	2209	161.4
H	0.118164063	1	121	
ERROR	0.000976563	1		

APPENDIX I: PAIRED T-TEST

Table I.1 t-test – Bench v.s. Pilot – BOD_5

15 cm Bench-scale RBF	30 cm Pilot-scale RBF
15	11
3	5
10	15
13	10
7	6
11	13

Paired t-Test: Paired Two Sample for Means

	15 cm Bench-scale RBF	30 cm Pilot-scale RBF
Mean	9.833333	10
Variance	18.56667	15.2
Observations	6	6
Pearson Correlation	0.654793	
Hypothesized Mean Difference	0.05	
df	5	
t Stat	-0.15472	
P(T<=t) one-tail	0.441548	
t Critical one-tail	2.015049	
P(T<=t) two-tail	0.883095	
t Critical two-tail	2.570578	

Table I.2 t-test – Bench v.s. Pilot – Turbidity

15 cm Bench-scale RBF	30 cm Pilot-scale RBF
4.3	2.9
1	2.6
1.3	3.2
1.2	2
1.3	1.7
2.2	3.5

Paired t-Test: Paired Two Sample for Means

	15 cm Bench-scale RBF	30 cm Pilot-scale RBF
Mean	1.883333	2.65
Variance	1.573667	0.483
Observations	6	6
Pearson Correlation	0.365899	
Hypothesized Mean Difference	0.05	
df	5	
t Stat	-1.6795	
P(T<=t) one-tail	0.076945	
t Critical one-tail	2.015049	
P(T<=t) two-tail	0.15389	
t Critical two-tail	2.570578	

Table I.3 t-test – Bench v.s. Pilot – NH_4^+-N

15 cm Bench-scale RBF	30 cm Pilot-scale RBF
1.1	0.9
1.1	1.1
0.6	0.9
0.6	1.6
0.8	1.3
0.9	0.6
0.8	0.6

Paired t-Test: Paired Two Sample for Means

	15 cm Bench-scale RBF	30 cm Pilot-scale RBF
Mean	0.842857	1
Variance	0.042857	0.133333
Observations	7	7
Pearson Correlation	-0.30867	
Hypothesized Mean Difference	0.05	
df	6	
t Stat	-1.16093	
P(T<=t) one-tail	0.144883	
t Critical one-tail	1.943181	
P(T<=t) two-tail	0.289767	
t Critical two-tail	2.446914	

Table I.4 t-test – Bench v.s. Pilot – TN

15 cm Bench-scale RBF	30 cm Pilot-scale RBF
15	13.4
13.8	15.6
12.4	16.2
13.8	17.2
13	12.4

Paired t-Test: Paired Two Sample for Means

	15 cm Bench-scale RBF	30 cm Pilot-scale RBF
Mean	13.6	14.96
Variance	0.96	3.988
Observations	5	5
Pearson Correlation	-0.19932	
Hypothesized Mean Difference	0.05	
df	4	
t Stat	-1.31735	
P(T<=t) one-tail	0.129056	
t Critical one-tail	2.131846	
P(T<=t) two-tail	0.258112	
t Critical two-tail	2.776451	

APPENDIX J: ANOVA

Table J.1 ANOVA – Ventilation – BOD_5

Top	Bottom Sidewall	Top & Bottom Sidewall
14	6	2
14	12	2
19	16	3
16	14	2
10	19	2
3	33	2
13	24	2
11		
2		
11		
7		
10		

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	852.163	2	426.0815	12.64578	0.000197	3.42213
Within Groups	774.9524	23	33.69358			
Total	1627.115	25				

Table J.2 ANOVA – Ventilation – Turbidity

Top	Bottom Sidewall	Top & Bottom Sidewall
1.5	1.2	1.6
2.6	6.6	3.5
1.0	3.6	1.5
0.9	3.6	3.6
3.9	3.9	5.0
1.0	1.4	2.5
0.9	1.0	2.1
1.0		1.6
0.9		2.9

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	10.67657	2	5.338287	2.670765	0.091537	3.443361
Within Groups	43.97328	22	1.998786			
Total	54.64986	24				

Table J.3 ANOVA – Ventilation – NH_4^+-N

Top	Bottom Sidewall	Top & Bottom Sidewall
0.8	0.9	1.8
0.9	0	0.6
1	1	2
0.1	0	1
0	0	0.4
0	1.8	0.6
0	0	
0	0.3	
2.1		

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	1.316618	2	0.658309	1.366417	0.27782	3.492829
Within Groups	9.635556	20	0.481778			
Total	10.95217	22				

Table J.4 ANOVA – Ventilation – Fecal coliforms

Top	Bottom Sidewall	Top & Bottom Sidewall
1.00E+01	1.30E+03	1.00E+03
4.30E+02	1.00E+01	6.20E+02
4.00E+02	6.00E+02	3.70E+02
1.00E+02	2.00E+02	1.00E+02
1.00E+02	1.00E+02	1.00E+02
1.00E+01	1.00E+01	1.00E+03
		1.00E+02

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	286523.7	2	143261.8	0.929838	0.414927	3.633716
Within Groups	2465150	16	154071.9			
Total	2751674	18				

Table J.5 ANOVA – Single Media – BOD_5

Crushed glass		geotextile	
low HLR	high HLR	low HLR	high HLR
8	26	9	17
14	21	11	29
14	9	16	22
17	11	11	25
15	20	13	25
6	18	16	
17	20	4	
2	13		
	10		
	22		
	7		
	17		

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	565.7628	3	188.5876	6.596806	0.001637	2.946685
Within Groups	800.456	28	28.58771			
Total	1366.219	31				

Table J.6 ANOVA – Single Media - Turbidity

Crushed glass		geotextile	
low HLR	high HLR	low HLR	high HLR
1.7	2.6	1.1	11.4
1.4	2.0	1.3	11.3
1.6	2.8	1.4	2.8
1.5	2.8	0.9	22.7
1.4	2.4	0.8	
	2.5	0.9	
	3.9		
	2.1		

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	354.5384	3	118.1795	11.09366	0.000197	3.127354
Within Groups	202.4047	19	10.65288			
Total	556.9432	22				

Table J.7 ANOVA – Single Media – NH_4^+-N

Crushed glass		geotextile	
low HLR	high HLR	low HLR	high HLR
0.3	0.0	0.3	3.6
1.0	0.0	1.5	4.8
1.3	1.0	0.8	9.0
0.5	0.0	0.5	0.0
1.0	0.0	0.8	33.1
0.0	1.3	0.3	12.5
	0.3		
	0.0		
	0.6		

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	462.8146	3	154.2715	4.986218	0.008257	3.027999
Within Groups	711.6106	23	30.93959			
Total	1174.425	26				

Table J.8 ANOVA – Single Media – Fecal Coliforms

Crushed glass		geotextile	
low HLR	high HLR	low HLR	high HLR
1.00E+02	4.50E+02	1.80E+03	6.00E+02
1.00E+02	1.10E+02	2.00E+03	1.50E+03
1.00E+03	3.00E+01	4.00E+03	3.30E+03
1.00E+03	1.00E+01	1.00E+02	1.00E+03
1.00E+03	1.00E+01	1.00E+02	1.00E+02
	5.00E+02		
	7.00E+01		
	3.60E+02		
	2.80E+03		
	3.50E+02		

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	5445126	3	1815042	1.597809	0.219852	3.072472
Within Groups	23855090	21	1135957			
Total	29300216	24				

Table J.9 ANOVA – Dual Media – BOD_5

3CG:1GT		2CG:2GT		CG:3GT	
Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR
2	14	4	28	9	17
2	18	9	25	12	30
6	12	13	49	14	15
2	10	2	21	2	35
2	2	25	79	14	51
2	2	4	55	3	11
	11		81		80
	16		67		57
	6				74
					46

Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	15809.65	5	3161.929	12.1469	3.9555E-07	2.455828
Within Groups	10152	39	260.3076			
Total	25961.64	44				

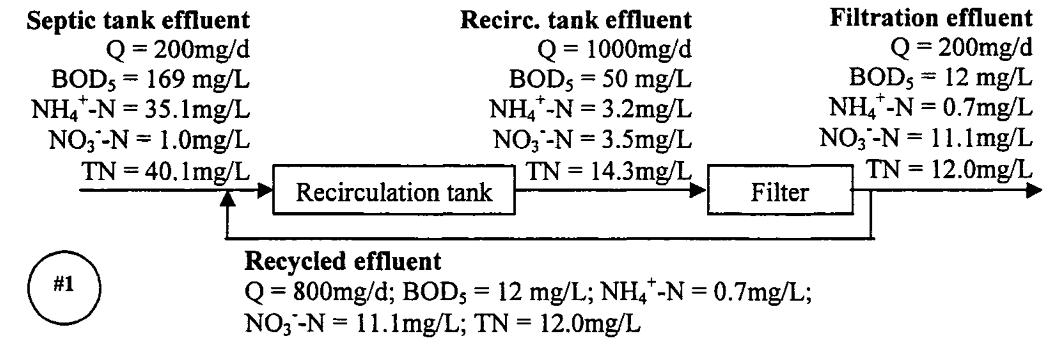
Table J.10 ANOVA – Dual Media – NH_4^+-N

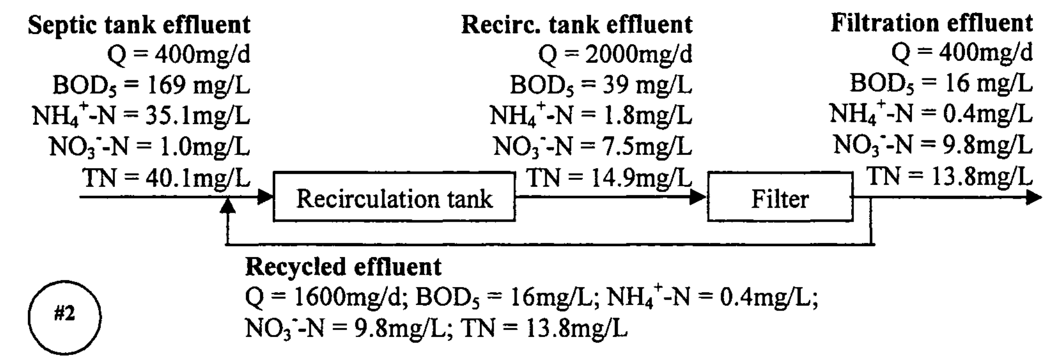
3CG:1GT		2CG:2GT		CG:3GT	
Low HLR	High HLR	Low HLR	High HLR	Low HLR	High HLR
0.1	0	0	1.4	0.8	1.8
0.4	0.6	0.4	2.4	13.6	4.5
0.3	1.6	0.8	1	10.3	11.5
0.1	1.4	0.1	16	7	12.8
1.3	0	0.1	4	3	1.6
0	4.9	0.6	29.1	4	18.1
0	4.3	0	16.5	4	15.3
	0		19.3		14.8
	0.8		13.1		13.8

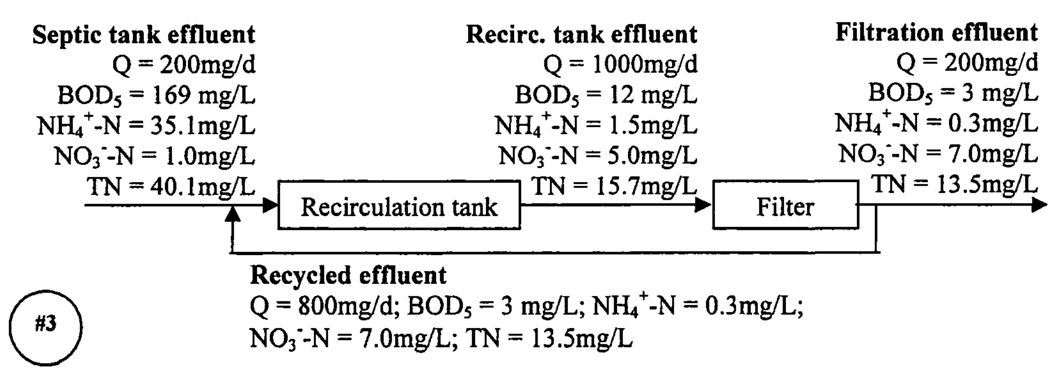
Source of Variation	SS	df	MS	F	P-value	F crit
Between Groups	1061.068	5	212.2136	7.260477	5.61365E-05	2.437694
Within Groups	1227.602	42	29.22861			
Total	2288.67	47				

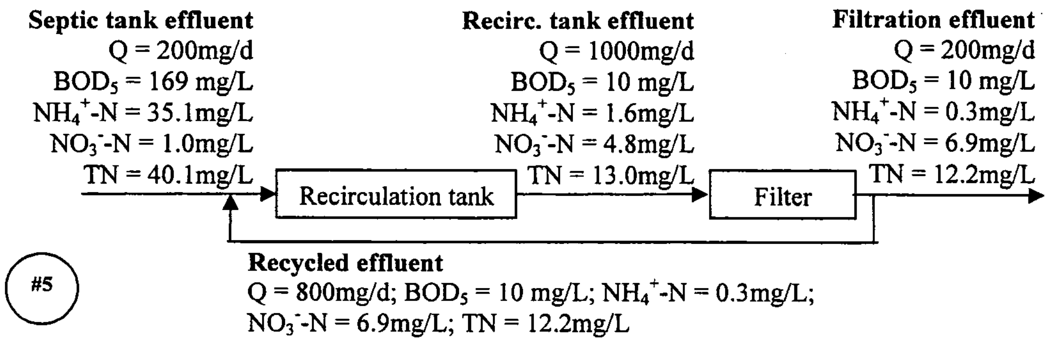
APPENDIX K: PROCESS OVERVIEW OF DUAL MEDIA RBFS

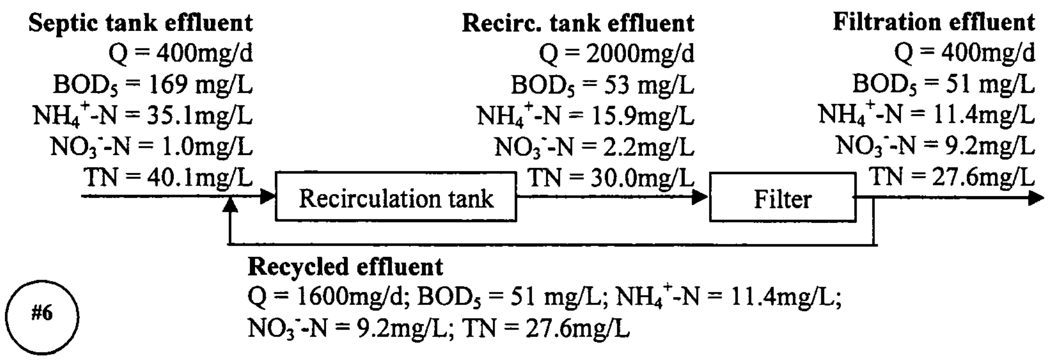
(Detailed information of each trial is listed in Table 7.1)

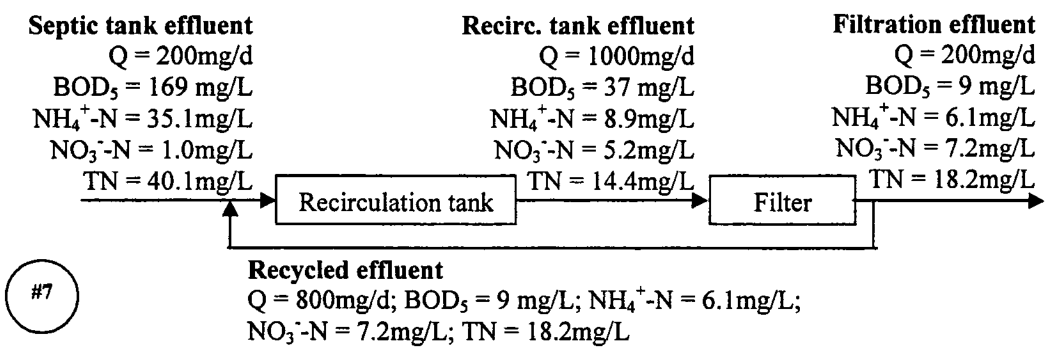







Septic tank effluent
Q = 200mg/d
BOD5 = 169 mg/L
NH4+-N = 35.1mg/L
NO3--N = 1.0mg/L
TN = 40.1mg/L
Recirc. tank effluent
Q = 1000mg/d
BOD5 = 10 mg/L
NH4+-N = 1.6mg/L
NO3--N = 4.8mg/L
TN = 13.0mg/L
Filtration effluent
Q = 200mg/d
BOD5 = 10 mg/L
NH4+-N = 0.3mg/L
NO3--N = 6.9mg/L
TN = 12.2mg/L
Recirculation tank
Filter
Recycled effluent
Q = 800mg/d; BOD5 = 10 mg/L; NH4+-N = 0.3mg/L;
NO3--N = 6.9mg/L; TN = 12.2mg/L
#5


Septic tank effluent
Q = 400mg/d
BOD5 = 169 mg/L
NH4+-N = 35.1mg/L
NO3--N = 1.0mg/L
TN = 40.1mg/L
Recirc. tank effluent
Q = 2000mg/d
BOD5 = 53 mg/L
NH4+-N = 15.9mg/L
NO3--N = 2.2mg/L
TN = 30.0mg/L
Filtration effluent
Q = 400mg/d
BOD5 = 51 mg/L
NH4+-N = 11.4mg/L
NO3--N = 9.2mg/L
TN = 27.6mg/L
Recirculation tank
Filter
Recycled effluent
Q = 1600mg/d; BOD5 = 51 mg/L; NH4+-N = 11.4mg/L;
NO3--N = 9.2mg/L; TN = 27.6mg/L
#6


Septic tank effluent
Q = 200mg/d
BOD5 = 169 mg/L
NH4+-N = 35.1mg/L
NO3--N = 1.0mg/L
TN = 40.1mg/L
Recirc. tank effluent
Q = 1000mg/d
BOD5 = 37 mg/L
NH4+-N = 8.9mg/L
NO3--N = 5.2mg/L
TN = 14.4mg/L
Filtration effluent
Q = 200mg/d
BOD5 = 9 mg/L
NH4+-N = 6.1mg/L
NO3--N = 7.2mg/L
TN = 18.2mg/L
Recirculation tank
Filter
Recycled effluent
Q = 800mg/d; BOD5 = 9 mg/L; NH4+-N = 6.1mg/L;
NO3--N = 7.2mg/L; TN = 18.2mg/L
#7