SUSTAINABLE DRINKING WATER TREATMENT FOR SMALL COMMUNITIES USING MULTISTAGE SLOW SAND FILTRATION

by

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I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners.

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Shawn A. Cleary

ABSTRACT

Slow sand filtration is a proven and sustainable technology for drinking water treatment in small communities. The process, however, is sensitive to lower water temperatures that can lead to decreased biological treatment, and high raw water turbidity levels that can lead to premature clogging of the filter and frequent cleaning requirements, resulting in increased risk of pathogen breakthrough.

Multistage filtration, consisting of roughing filtration followed by slow sand filtration, can overcome these treatment limitations and provide a robust treatment alternative for surface water sources of variable water quality in northern climates, which typically experience water temperatures ranging down to 2^{o}C. Prior to this study, however, multistage filtration had yet to be systematically challenged in colder climates, including testing of its performance under increased hydraulic loadings and elevated influent turbidity together with cold water conditions.

The primary goal of this research was to demonstrate the reliability of multistage filtration for small communities in northern climates with reference to the Ontario Safe Drinking Water Act. In this research, testing was conducted on two different pilot multistage filtration systems and fed with water from the Grand River, a municipally and agriculturally impacted river in Southern Ontario. One system featured pre-ozonation and post-granular activated carbon (GAC) stages, and shallower bed depths in the roughing filter and slow sand filter. The other system featured deeper bed depths in the roughing filter and slow sand filter, two parallel roughing filters of different design for comparison, and a second stage of slow sand filtration for increased robustness.

Removal of turbidity, total coliforms, and fecal coliforms under a range of influent turbidities (1 to >100 NTU), water temperatures (~2 to 20^{o}C), and hydraulic loading rates (0.2 to 0.8 m/h) were investigated. In addition, the slow sand filters in each pilot system were challenged with high concentrations (~10^{6} oocyst/L) of inactivated *Cryptosporidium parvum* oocysts.

The performance of both pilot multistage filtration systems was highly dependent on the biological maturity of the system and its hydraulic loading rate. In a less mature system operating in cold water conditions (<5^{o}C), effluent turbidity was mostly below 0.5 NTU during periods of stable influent turbidity (no runoff events) and a hydraulic loading of 0.4 m/h. However, runoff events of high influent turbidity (>50 NTU), increased hydraulic loadings (0.6 m/h), and filter cleaning occasionally resulted in effluent turbidity above 1 NTU. Furthermore, in a less mature system operating during runoff events of high turbidity, reducing the hydraulic loading rate to 0.2 m/h was important for achieving effluent turbidity below 1 NTU.

In a more mature system operating in warm water conditions (19-22^{o}C), effluent turbidity was consistently below 0.3 NTU at a hydraulic loading rate of 0.4 m/h, and below 0.5 NTU at 0.8 m/h, despite numerous events of high influent turbidity (>25 NTU). It remains to be seen whether this performance could be sustained in colder water temperatures with a fully mature filter.

Removal of coliform bacteria was occasionally incomplete in a less mature system, whereas, in a more mature system operating in warm water conditions (>9^{o}C), removal was complete in all measurements. Furthermore, the average removal of *Cryptosporidium* was greater than 2.5 logs in both systems (with hydraulic loading rates ranging from 0.4 to 0.8 m/h) and improved with increased filter maturity.

Each individual stage of the multistage system was an important treatment barrier in the overall process of turbidity and pathogen removal. The roughing filter was not only important for protecting the slow sand filter from solids loading and increasing its run length, but was also a significant contributor to coliform removal when the system was less mature. Removal of turbidity was significantly improved when the roughing filter was more mature, suggesting that biological treatment was an important treatment mechanism in the roughing filter. Although pre-ozonation was used mainly for the removal of organic carbon and colour, it achieved complete removal of coliform bacteria and was also suspected to be important for enhanced removal of turbidity. The second slow sand filter in series provided additional robustness to the process by reducing effluent turbidity to below 1 NTU during cold water runoff events of high turbidity and increased hydraulic loadings (0.6 m/h), while achieving effluent below 0.3 NTU during normal periods of operation. It also provided additional removals of coliforms under challenging operating conditions, and contributed an additional 0.8 logs average removal of *Cryptosporidium*, which resulted in cumulative removal of 3.7 logs, approximately 1 log greater than all the other challenge tests.

Collectively, the entire multistage system performed well with water temperatures ranging down to 2^{o}C, limited filter maturity, elevated raw water turbidities, and increased hydraulic loading rates. Its ability to meet the current Ontario turbidity regulations and greater than 2 log removal of *Cryptosporidium* over a range of operating conditions, with little or no process adjustment, is a testament to the robustness and minimal maintenance requirements of the process, which are desirable attributes for small water systems that are often located in rural areas. While this research demonstrated the performance of multistage filtration using pilot scale testing, it is important to note that full-scale plants tend to produce significantly better results than pilot facilities, due to long term biological maturation of the system.

Overall, multistage filtration is a sustainable and cost-effective technology that, through this research, appears to be a safe, reliable, and robust treatment alternative for small and non-municipal water systems in North America and the developing world. Further, based on its performance with challenging influent water quality and cold water conditions, multistage filtration holds particular promise for small communities in northern climates that are required to meet safe drinking water regulations, but are dependent on surface water sources of variable water quality and temperatures.

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1 Introduction

1.1 THE BENEFITS OF MULTISTAGE FILTRATION AND THE NEED FOR RESEARCH IN NORTHERN CLIMATES

In North America and around the world, there is a significant need for sustainable, cost-effective, and reliable drinking water treatment alternatives that provide a safe water supply in small communities. Water supply surveys have shown that the "rate of non-compliance with drinking water standards increases in proportion to decreases in the size of the population served" (Lippy and Waltrip, 1984). This is likely because many conventional treatment technologies produce less-consistent performance when scaled down, are relatively maintenance intensive, and lack economies of scale, such that the cost per unit volume of treated water is higher in smaller systems. Many small communities in North America are faced with the challenge of compliance as 92% of all water systems in the United States serve a population fewer than 5,000 people (AWWA, 1981). Canada likely has a similar percentage of small water systems.

Slow sand filtration is a proven, sustainable, and reliable drinking water treatment alternative for small communities, which can be beneficial for addressing the small systems challenges described above. The process provides treatment through physical filtration of particles and biological removal of pathogens and organics in the upper biologically active layer of the sand bed, known as the schmutzdecke. It is a simple technology that is relatively easy to operate and understand, requires little maintenance, and is capable of achieving high standards of treatment without the use of coagulant chemicals, of which the purchase (material) and transport costs alone can be debilitating for a remote community. Furthermore, the performance of chemically assisted filtration is highly dependent on source water chemistry, and can be operationally intensive for surface water of variable water quality, in terms of maintaining optimal coagulant dosage and pH levels.

Slow sand filtration, on the other hand, can operate under a wide range of operational and source water quality conditions, with little or no process adjustment, which is a testament to the robustness of the process. It also requires less maintenance in terms of cleaning, as typical filter run lengths of one or more months can be expected between cleanings. In addition to the cost savings from reduced maintenance, its operational energy costs are minimal, as water can flow through the filter under gravity conditions, without the use of pumping, and energy-intensive backwashing of the filter is not required in the cleaning process (for more details of the cleaning process, refer to Section 2.3.4). Finally, another advantage of slow sand filtration for a remote community is that it does not require the use of specialized equipment and can potentially be constructed with locally available materials, mainly from properly graded sand/gravel, concrete, and standard piping.

However, while slow sand filtration presents a number of attractive advantages over other conventional technologies, it does have a few significant limitations, which may be of concern for some source waters. First of all, the process is sensitive to raw water turbidity where consistently high levels of turbidity (20 to 30 NTU) can lead to premature clogging of the filter, decreased filter run length, and frequent cleaning requirements. Besides increased maintenance requirements, frequent cleaning also results in an increased opportunity for pathogen breakthrough by disturbing the biological equilibrium of the filter bed. Secondly, treatment is less efficient at low water temperatures ($<5^{o}C$), which hinder biological activity in the filter bed. This poses a particular concern for communities in northern climates. Finally, it is less efficient than chemically assisted filtration at removing negatively charged stable suspensions of colloidal matter (most colloidal material in natural water is negatively charged).

Multistage filtration, consisting of a slow sand filter preceded by a roughing filter, can overcome these treatment limitations and provide a reliable treatment alternative for surface water sources of variable turbidity and cold temperatures. It is a robust multi-barrier treatment process, in which waterborne particulate matter and pathogens face a series of treatment barriers, which may produce an effluent water quality that is better than that of slow sand filtration alone. The roughing filter, consisting typically of three layers of gravel media

(coarse to fine), is used to attenuate turbidity peaks, especially during runoff events, and reduce the solids loading on the slow sand filter, hence increasing its run length, and improving effluent quality. The roughing filter may also allow the slow sand filter to operate at higher than typical hydraulic loading rates. Furthermore, it provides a large surface area for the sedimentation of colloidal matter, and provides additional biological treatment and hydraulic retention time, which may be important during colder water temperatures and runoff events of highly contaminated source water. Thus, roughing filters may not only be important for protecting the operational integrity of the slow sand filter, but may also be a significant treatment barrier in the overall process of pathogen removal. But most importantly, the roughing filter does not require energy for backwashing, thus it is consistent with the cost-sustainable paradigm that traditional slow sand filtration is reputed for.

The success of multistage filtration has been documented in several jurisdictions throughout the world, mainly in tropical regions using extremely contaminated surface water with high solids loadings. In North America, where single stage slow sand filtration has been largely replaced by chemically assisted rapid sand filtration, there has been a recent increase of interest in the use of multistage filtration for surface water treatment. LeCraw et al. (2004) document the performance of multistage filtration in a number of pilot and full-scale installations in cold climate locations throughout North America. The results are promising for small communities in northern climates that are seeking a safe and cost-effective treatment alternative.

However, multistage filtration has yet to be systematically challenged in colder climates. There are still a number of key issues that need to be investigated to define the envelope in which it can perform satisfactorily for small systems in North America. Such issues include its performance under increased hydraulic loadings and elevated influent turbidity together with cold water conditions. Recent literature reveals the impact of additional stages of pre-ozonation and post-granular activated carbon filtration for removal of organics, colour, and disinfection by-product precursor material; however, further exploration under the above operating conditions is warranted. Furthermore, the impact of an additional slow sand filter in series for increased robustness and protection against pathogen breakthrough, particularly

during runoff events, cleaning, decreased temperatures, and increased hydraulic loadings remains to be determined. An investigation into increasing the filtration efficiency of roughing filtration, without compromising its operational benefits, may also be important for branding the roughing filter as a significant treatment barrier in the overall process of pathogen removal, beyond just its role of increasing the filter run length of the slow sand filter.

Most importantly, if this technology is to be legitimately accepted as a small system treatment alternative, it is important to define the range of operating conditions under which it could comply with the requirements of regulations such as the Ontario Safe Drinking Water Act, specifically with respect to the removal of turbidity, *Giardia lamblia*, and *Cryptosporidium parvum.* (Given that systems in Ontario would also include disinfection, bacterial removals by multistage filtration, while important, are not critical from a regulatory perspective). By investigating its performance under a range of raw water turbidity, temperatures, and hydraulic loadings, a range of small system conditions could be established, for which multistage filtration is a viable drinking water treatment option. This kind of research would further elucidate the potential of multistage filtration as a reliable, cost-effective, and robust drinking water treatment technology for small communities throughout the world, especially in northern climates.

1.2 RESEARCH OBJECTIVES

The primary goal of this research was to demonstrate the reliability of multistage filtration for small communities in northern climates with reference to the Ontario Safe Drinking Water Act.

Specific objectives to meet this goal were to:

1. Determine the performance of multistage filtration, particularly with respect to the removal of turbidity and coliform bacteria, with increased raw water turbidity (up to and above 100 NTU), low water temperatures (below 5^{o}C), and increased hydraulic loadings (up to 0.8 m/h).

2. Determine the removal efficiency of *Cryptosporidium* oocysts by challenging the slow sand filter with influent concentrations of approximately 10^6 oocysts/L at a range of hydraulic loading rates (0.4 to 0.8 m/h).
3. Determine the importance of the roughing filter as a significant treatment barrier in the overall process of pathogen removal, beyond its traditional role of protecting the operational integrity of the slow sand filter.
4. Determine the impact of pre-ozonation and post-granular activated carbon stages on the removal of turbidity and coliform bacteria.
5. Determine the impact and added robustness of two slow sand filters in series.

1.3 RESEARCH APPROACH

After a thorough literature review, research needs were defined and two pilot systems were commissioned from October to December 2003. One of the systems was on loan for the study and the other was constructed particularly for this study. Both pilots were fed with raw water from the Grand River in Southern Ontario, and systematic testing took place from February to June 2004, when a wide range of seasonally affected water quality conditions occurred. The influent turbidity varied from 1 to 100 NTU, water temperature varied from 2 to 20^{o}C, and hydraulic loading rates were varied from 0.2 to 0.8 m/h. Water samples were collected from sampling ports throughout each pilot system and results were analyzed to determine performance under a wide range of operational and water quality conditions. The results offer insight to the benefits of multistage filtration in providing a safe, reliable, and robust drinking water supply in small and rural communities.

1.4 THESIS ORGANISATION

Chapter 2 is a literature review which presents background information on traditional slow sand filtration, its virtues and limitations, followed by recent advances and developments on this technology. This is followed by information on the benefits and performance of multistage filtration, particularly roughing filtration, and its success in tropical areas and recently in North America. Chapter 3 describes the design of the pilot apparatus used in this study, the experimental design, including testing schedules and operational conditions, and

the analytical methods employed throughout this study. Chapter 4 presents background information on the Grand River and its appropriately challenging water quality conditions for testing the pilot multistage filtration systems. Chapter 5 presents the effluent turbidity results from the online turbidity measurements, which demonstrate the performance of the entire multistage filtration system under a wide range of operating and water quality conditions. This is followed by an analysis of the hydraulic data, headloss development, and filter run length during the study. Chapter 6 presents the results from the manual handheld turbidity measurements before and after each stage in both pilot systems, which demonstrates the performance of individual stages in the multistage process. Results from the total and fecal coliform measurements throughout the process are discussed, as well as the results from the *Cryptosporidium* challenge tests. Finally, Chapter 7 presents the conclusions of this research, practical recommendations for full-scale installations, and recommendations for future research.

Chapter 2

2 Literature Review and Background

2.1 THE GLOBAL NEED FOR CLEAN WATER

A common definition of clean water is water that is free of pathogenic organisms, toxic substances, colour, turbidity, taste, and odour, and an acceptable level of minerals and organic material (Thanh and Hettiaratchi, 1982). Every human on our planet has a fundamental right to a reliable supply of clean water. Yet, according to the World Health Organization, there are still 1.1 billion people in the world without access to an improved water supply (WHO and UNICEF, 2000). This translates to 6% of the global population lacking access in urban areas, and 29% lacking access in rural areas. This is not only a critical problem in developing countries, but also a challenge faced by many municipalities in both rural and remote areas of the developed world.

The results of inadequate water supply are catastrophic, as 2.2 million deaths related to diarrhoeal disease occur every year, which equates to one water-related death every 15 seconds (WHO and UNICEF, 2000). Thus, there is a global need for clean water and every man, woman, and child has a fundamental right to a reliable supply.

2.2 THE PROBLEM WITH SURFACE WATER SUPPLY IN RURAL AREAS

It is estimated that 70% of the world's population live in rural areas (Thanh and Hettiaratchi, 1982). In rural areas of both developed and developing countries, many communities rely on surface water (rivers, lakes, etc.) as a source for drinking water. Groundwater resources may be scarce due to the natural geology of the area, lack of rainwater infiltration in overdeveloped areas, and overuse or contamination of groundwater resources. Often, small communities "do not have the power or the resources to protect watersheds and frequently rely on poor water sources" (LeCraw et al., 2004).

Sims and Slezak (1991) surveyed a number of rural water treatment facilities serving mostly communities with less than 10,000 people. 54% of the facilities used streams as the raw water source, 41% used lakes, and only 5% used groundwater. Thus, surface water is most often the source for rural drinking water supply.

Yet, often the surface water source represents a threat to human health due to natural and/or anthropogenic contamination. Naturally occurring influences, such as seasonal precipitation patterns and runoff events, results in fluctuations of turbidity, nutrients, and suspended solids. Anthropogenic influences include industrial pollution, municipal wastewater treatment discharge, improperly designed septic systems and latrines, improper management of agricultural drainage, excessive or improper application of excreta-based agricultural fertilizer, and the culture of livestock in direct vicinity of the surface water. However, it is the combination of both naturally occurring influences with anthropogenic influences, which compound to have the largest impact on surface water quality.

This impact has been experienced globally and throughout North America, where widespread contamination of surface water has led to an emergence of parasitic *Giardia* and *Cryptosporidium* as a major cause of waterborne disease in humans. In fact, *Giardia* is the most commonly reported intestinal protozoan infection worldwide (USEPA, 1999). The World Health Organization has estimated that 200 million people are infected with *Giardia* every year. In the U.S., it is one of the leading causes of disease outbreaks (Clark and Regli, 1991), and the most frequently identified etiological cause of waterborne outbreaks in public water systems (USEPA, 1999). Furthermore, it is estimated that on average, 1 to 6% of the North American population is inflected with *G. lamblia* (Bryck and Sklenar, 1986).

In 1984, *Cryptosporidium* became widely recognized as a harmful waterborne pathogen during an outbreak of more than 2000 infected individuals in Texas (D'Antonio et al., 1985). Since its recognition, it is estimated to have affected more than one million individuals in Europe and North America. The largest reported outbreak occurred in Milwaukee in 1993 when 400,000 people became infected from the local water supply.

Most importantly, it is the small communities that are at the highest risk of water-related outbreaks. In fact, water supply surveys have shown that the "rate of non-compliance with drinking water standards increases in proportion to decreases in the size of the population served" (Lippy and Waltrip, 1984). This is because small water systems often lack economies of scale and simply scaling down existing conventional technologies may not be operationally or economically feasible. This is clearly a growing concern for many communities in North America where 92% of all water systems in the United States serve a population of fewer than 5,000 people (AWWA, 1981).

2.2.1 What are *Giardia* and *Cryptosporidium*?

Giardia and *Cryptosporidium* are protozoan parasites than can live in the intestines of humans and animals. *Giardia* is in the form of a chlorine resistant cyst approximately 7-12 μm in size and can be found even in pristine surface waters, and in locations ranging from the arctic to the tropics (USEPA, 1999). However, *Giardia* levels are higher in areas with agricultural activity and municipal wastewater discharge (USEPA, 1999). One of the most infective forms of *Giardia* in humans is *Giardia lamblia (G. lamblia)*, and the effective dose can be as little as 1 to 10 cysts (Bryck and Sklenar, 1990).

Cryptosporidium is in the form of an environmentally durable and chlorine resistant oocyst approximately 4-6 μm in size. In fact, *Cryptosporidium* oocysts are 30 times more resistant to ozone and chemical disinfectants than *Giardia* (Huck et al., 2001). The number of oocysts typically found in a contaminated water source ranges from 10-10,000 oocysts/L (Sattar et al., 1999). Among the several species of *Cryptosporidium*, only one species, *Cryptosporidium parvum (C. parvum)*, is infective in humans and can be life threatening for immune-compromised individuals. After ingestion, the parasite emerges from the oocyst and infects the lining of the intestine (Pontius, 1995). Depending on immunity of the host, illness can result from ingesting as little as 1 oocyst or up to 100 oocysts.

2.2.2 Coping with *Giardia* and *Cryptosporidium* in Drinking Water

An analysis of surface water samples taken from major U.S. watersheds indicated that *Giardia* and *Cryptosporidium* were present in 45% and 60.2% of samples, respectively (LeChevallier and Norton, 1995). In these samples, the concentration of *Giardia* cysts varied from 2 to 4,380/100 L. This variation is not only seen across a number of watersheds, but within a single watershed LeChevallier and Norton (1992) reported that "*Giardia* and *Cryptosporidium* levels may vary as much as 600-fold".

Even in filtered water supplies, LeChevallier and Norton (1992) found that 37% of all samples tested positive for *Giardia* and *Cryptosporidium*, however, most were non-viable. Although, in a similar study, Aboytes and LeChevallier (2003) analyzed effluent from 82 conventional surface water treatment plants and found that 1.4% of the samples contained viable *Cryptosporidium*. Even more disturbing is that 70% of all positive samples occurred in filtered water with turbidity less than 0.1 NTU, and 20% with turbidity less than 0.05 NTU. Furthermore, 58.3% of all positive samples occurred between April and June, when surface runoff was an influential factor.

Overall, throughout the U.S., 103 outbreaks between 1965 and 1996 have been attributed to contaminated drinking water from public water systems (USEPA, 1999). In response to this nationwide epidemic, the U.S. Environmental Protection Agency (USEPA) initiated the Surface Water Treatment Rule (SWTR), which was prompted by amendments to the Safe Drinking Water Act in 1996. This was soon followed by the Interim Enhanced Surface Water Treatment Rule (IESWTR) and the Long Term I Enhanced Surface Water Treatment Rule. These regulations stipulate a requirement for the removal of 99.99% of enteric (intestinal sourced) viruses from drinking water, 99.9% removal of *Giardia*, and 99% removal of *Cryptosporidium* (USEPA, 1999; USEPA, 2001).

2.2.3 The Importance of Filtration

The importance of filtration in meeting the requirements of the SWTR has not been learned without incurring tragic costs to community health. In fact, 52 of the 108 reported outbreaks

between 1965 and 1996 were caused by inadequate treatment of surface water, where chlorination was used, but not filtration (USEPA, 1999). This is because cysts and oocysts are highly resistant to chemical disinfectants such as chlorine. Furthermore, Craun et al. (1994) found that the incidence of water related disease (mostly *Giardia* related) was 8 times lower for communities using filtration, compared to those using only chlorination. In response, the USEPA recommends that a multi-barrier treatment approach, including filtration and disinfection under optimal operating conditions, can protect against waterborne transmission of *Giardia* and *Cryptosporidium* (USEPA, 1999; USEPA, 2001).

2.3 BACKGROUND ON SLOW SAND FILTRATION

2.3.1 The Promise of Slow Sand Filtration for Rural Areas

Slow sand filtration has been recognized as an appropriate technology for drinking water treatment in rural areas, and is recognized as a suitable filtration technology for removing waterborne pathogens. It is capable of improving the physical, chemical, and microbiological quality of water in a single treatment process without the addition of chemicals, and can produce an effluent low in turbidity and free of bacteria and viruses. In fact, Wegelin (1988) states, "no other single water treatment process can improve the physical, chemical, and bacteriological water quality of surface water better than slow sand filtration". In addition, the USEPA (1997) states, "when used with a source water of appropriate quality, slow sand filtration may be the most suitable filtration technology for small systems". These two statements elucidate the important role of slow sand filtration for treating surface water in small systems.

Slow sand filtration is a sustainable technology for rural water treatment because it is low cost and simple to operate. In addition, it is able to produce excellent effluent quality without the use of treatment chemicals. In fact, under good source water conditions, Cleasby et al. (1984a) found that slow sand filtration achieved better treatment than coagulation followed by direct filtration. In addition to the potential health hazard of long-term chemical exposure, treatment chemicals are also costly to manage in rural water systems. Due to lack of availability in rural areas, the transportation costs of importing chemicals can be a major

concern for small systems. In addition, the use of chemicals requires more maintenance and monitoring from skilled personnel, as the chemical dosing process is highly sensitive to fluctuations in raw water quality such as pH. Thus, the on-going operational costs of a conventional treatment system that uses chemicals can be overwhelming for a small community.

Slow sand filters can be constructed from local materials, can operate without the use of specialized equipment, and is much less labour intensive than rapid filters. Also, as slow sand filters operate under gravity flow conditions and energy intensive backwashing is not required, its on-going energy demand is minimal. It is important to understand that "while construction and capital costs are often paid for or subsidized by senior levels of government, it is the local community that must pay operations costs". Thus, slow sand filtration is an attractive treatment alternative for local communities.

Finally, there is very little water wastage during cleaning of the filters and the production of sludge is much less than rapid sand filters. The sludge can subsequently be handled in its dry state, preventing recontamination of surface water; and used as an amendment to agricultural fertilizer (Huisman and Wood, 1974).

2.3.2 Brief History of Slow Sand Filtration

Slow sand filtration dates back to 1804 in Paisley, Scotland, where John Gibb supplied water to the city from the slow sand filter at his bleachery (Baker, 1948). However, the current model for slow sand filtration originated from a one-acre slow sand filter designed by James Simpson for the Chelsea Water Company in London in 1829, which treated surface water from the Thames River (Barrett et al., 1991). Shortly after, filtration became a legal requirement in London for all surface waters (Baker, 1948). But it was not until 1892, during the advent of the germ theory of disease, that the effectiveness of slow sand filtration in preventing waterborne disease was discovered. At this time, there was a cholera epidemic in the city of Hamburg, Germany, where more than 7,500 people died. However, the

neighbouring city of Altona, which treated its water with slow sand filtration, had only a few deaths resulting from the epidemic.

The first slow sand filter in the United States was designed by James Kirkwood and completed in 1872 (Baker, 1948). This was followed by many more installations. However, due to dramatically reduced filter run times when treating high turbidity surface water, slow sand filtration has since been largely replaced by rapid filtration. However, there has been a recent re-emergence of slow sand filtration in small communities due to its effectiveness in removing natural organic matter and pathogens such as *Giardia* (Brink and Parks, 1996). In the state of New England alone, there have been over 25 new facilities either piloted or constructed since 1988. Currently, the USEPA recognizes slow sand filtration as an acceptable water treatment technology, which provides safe water for human consumption.

2.3.3 Brief Description of Slow Sand Filtration

The essential parts of a slow sand filter are shown in Figure 2.1. It consists of filter box with approximately 1 m depth of sand media. The recommended effective diameter of the media is between 0.15 and 0.35 mm and the uniformity coefficient (UC) should be between 1.5 and 3 (Huisman and Wood, 1974). Water is fed into the top of the filter and flows downward through the sand at a rate of 0.1 to 0.4 m/h. Organic and inorganic particulate matter and pathogenic microorganisms are removed by physical filtration and biological degradation in the sand bed. Further explanation of the physical and biological treatment mechanisms is given in section 2.3.5 and 2.3.6, respectively.

Most of the treatment occurs at the top of the sand bed where deposits of particulate and algal matter, combined with the dense growth of biomass, form a surface layer commonly known as the 'schmutzdecke'. However, significant additional treatment also occurs throughout the rest of the sand bed. The sand bed is supported by layers of underdrain gravel or support gravel, which prevent sand from clogging the underdrain piping and allow filtrate to drain freely from the filter. An adjustable weir downstream of the filter controls the water level of the supernatant above the filter.

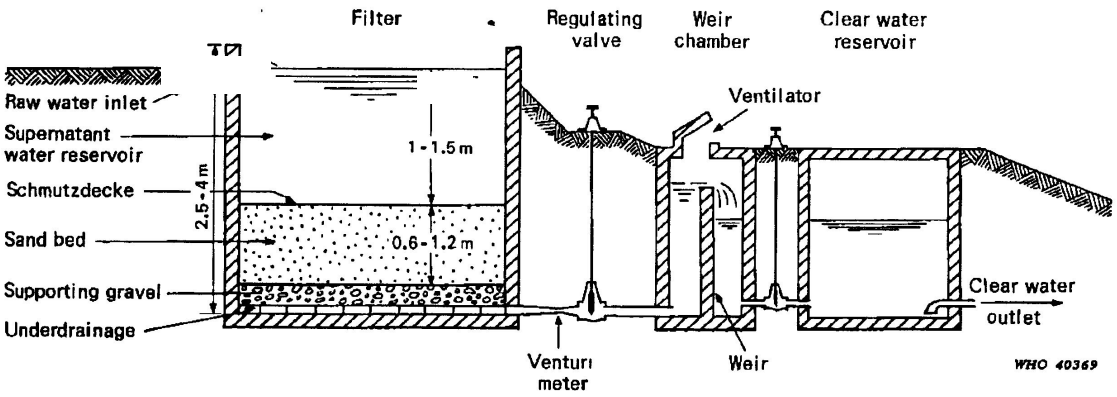

Figure 2.1: Diagram of a Slow Sand Filter

(Reprinted by permission of the World Health Organization, from Slow Sand Filtration, by Huisman, L. and Wood, W.E., WHO, Geneva, Switzerland, 1974)

The literature reveals some variation in the recommended design parameters for slow sand filters. Galvis et al. (2002) compiled a summary of the recommended design parameters from various authors. A condensed version of this is shown in Table 2.1.

Table 2.1: Recommended Design Parameters for Slow Sand Filters

Design Criteria	**Recommendations**		
	Ten States Standards USA (1987)	**Huisman and Wood (1974)**	**Visscher et al. (1987)**
Bed depth (m)	0.8	1.2	0.9
Effective media size (mm)	0.3 - 0.45	0.15 - 0.35	0.15 - 0.3
Uniformity coefficient	<2.5	<2	<3
Filtration rate (m/h)	0.08 - 0.24	0.1 - 0.4	0.1 - 0.2
Support bed (m)	0.4 - 0.6	not reported	0.3 - 0.5
Max. height of supernatant water (m)	0.9	1 - 1.5	1

Adopted from Galvis et al. (2002)

2.3.4 Cleaning of Slow Sand Filters

Cleaning must be performed at the end of a filter run, when the headloss across the filter bed has reached its maximum. This occurs when the water level above the sand bed reaches the overflow and begins to flow to waste, thus reducing the actual flow rate through the sand bed. Typically, filter run times range from 30 to 60 days, but could reach more than 100 days (Ellis, 1985).

The rate of headloss development is dependent on the filtration rate, the size and uniformity coefficient of the media, and the water quality conditions (Ellis, 1985). For example, a higher filtration rate delivers a higher solids loading to the filter and deposition of particulate matter occurs at a faster rate. In addition, headloss is proportional to the velocity through the sand bed, thus as filtration rate increases, headloss increases.

The traditional method of cleaning slow sand filters involves draining the water level down to just below the sand surface and scraping off the top 1 or 2 cm of schmutzdecke. The schmutzdecke is where the highest concentration of biomass exists, hence the region where most biological treatment is achieved. Thus, pathogen removal may be compromised for a couple days after cleaning until biofilm maturity is re-established. In some cases, however, cleaning may have no effect on treatment efficiency. For example, Fox et al. (1984) found that bacteria removal was unaffected by scraping, and Poynter and Slade (1977) found that scraping had little effect of the removal efficiency of viruses.

After cleaning, the filters are refilled with water from the bottom of the filter. This is done to avoid entrapment of air bubbles and prevent the scouring of the sand bed if the filter were filled from the top.

Eighmy and Collins (1988) reported using an alternative method of cleaning known as "harrowing" where the sand is raked by a comb harrow, which penetrates 30 cm into the sand bed and detaches particulate debris. The debris is then washed away by a continuous flow of water across the top of the sand bed.

Generally, cleaning times are significantly lower with the harrowing method than the scraping method, and filters could be put back online within days instead of weeks. Also, this method results in minimal or no sand loss, thus re-sanding of the filter after many years of operation is not an issue. But most importantly, Eighmy and Collins (1988) found that very little biomass was lost during cleaning and biomass populations penetrated deeper into the sand bed, providing more biological contact time and improving removals of non-purgeable dissolved organic carbon (NPDOC), ultraviolet (UV) absorbance, and trihalomethane formation potential (THMFP). A description of these parameters is provided in section 2.4.

An additional advantage of harrowing is that it is an *in situ* cleaning method, and it is not necessary to drain the water level down to expose the sand. Lloyd (1996) found that some protozoa such as spirotichs, which graze on incoming bacteria, are particularly susceptible to desiccation when the sand is exposed. Thus, *in situ* methods of cleaning are preferred to maintain the viability of the biomass ecosystem in the sand bed.

2.3.5 Physical-Chemical Mechanisms of Removal in Slow Sand Filtration

Physical-chemical mechanisms of filtration are divided into two categories: transport mechanisms and attachment mechanisms. Transport mechanisms govern the transport of particulate matter to the filter media (otherwise referred to as collectors) and attachment mechanisms govern the attachment of particles to the media.

One of the major types of transport mechanisms in slow sand filtration is straining or screening, where particles that are larger than the pore size of the media are physically removed. Huisman and Wood (1974) approximated the pore size of a given media to be about 15% of the media diameter. Thus, it is feasible that a 0.2 mm diameter media could strain out particles larger than 30 μm in size (Haarhoff and Cleasby, 1991). However, as the pore size of the media progressively decreases due to particle deposition and biofilm growth, straining will become more efficient in capturing particles that are even smaller in size (Weber-Shirk and Dick, 1997b).

There are particles in surface water that are much smaller than the pore size of the media, such as bacteria (0.1 to 10 μm), viruses (0.01 to 0.1 μm), and colloidal particles (0.001 to 1 μm) (Montgomery, 1985). These particles penetrate deeper into the sand bed, where other mechanisms of transport become important. Impaction occurs when the inertia of the particle approaching the collector is greater than the hydrodynamic force that is carrying the water past the collector (Montgomery, 1985). The particle will deviate from the flow path and impact the collector. Hydrodynamic forces that result from non-laminar regions of flow, changes in flow velocity, and change of pore size may also transport particles to the surface of the collector (Montgomery, 1985).

Sedimentation occurs when the mass density of a particle is much greater than that of water and its settling velocity causes the particle to deviate from the flow path and settle onto the media surface. Ellis (1985) reports that sedimentation is probably more important with suspended particulates between 4 and 20 μm in size.

Interception occurs when deposited particles accumulate on the media surface, gradually reduce the pore size, and act as additional collectors for subsequently passing particles. It is generally known that as the ratio of the particle size to media size increases, interception also increases (Montgomery, 1985).

Particles in the colloidal range (less than 1 μm in diameter) are influenced by Brownian motion or diffusion and will deviate from flow paths toward the filter media, depending on the electrostatic interaction between the particles and the media (Montgomery, 1985).

As particles are transported to the filter media, attachment mechanisms will act to capture the particle resulting in a successful collision. Such attachment mechanisms include mass attraction (Van der Waals force) and electrostatic attraction between oppositely charged particles (Montgomery, 1985). The effects of Van der Waals forces, however, are only significant if the particle can overcome any electrostatic repulsion barrier and reach the surface of the media (Haarhoff and Cleasby, 1991). Interestingly, McConnell (1984) suggests the possibility of multivalent cations acting as a bridge between negatively charged surfaces

and negatively charged particles. This theory was confirmed by the finding that "virus adsorption on sand is enhanced with increasing ionic strength and with higher concentrations of higher valence cations in solution" (Galvis et al., 1998).

Adsorption of particles to the media is another important attachment mechanism. Microorganisms such as algae and bacteria will colonize the filter bed and form a sticky zoogleal biofilm on the sand grains to which particles can become attached to. Ellis (1985) suggests that adsorption is more important for smaller particles.

Detachment of particles is another important phenomenon of filters. As particle deposits and growth of biofilm reduce the pore size of the media, the interstitial velocity in the pores increases. This causes an increase in the hydrodynamic shear force on particle deposits and may cause particles to become detached. Shearing forces are expected to be highest in the schmutzdecke (Weber-Shirk and Dick, 1997b). Increased detachment may also occur with sudden increases in the flow rate or influent solids concentrations.

Detached particles can then penetrate deeper into the filter bed and may ultimately breakthrough the filter. For example, Ellis and Aydin (1995) found that particulate deposits decreased rapidly with depth, however were still present at a depth of 400 mm. This highlights the importance of maintaining consistent operational conditions, and avoiding sudden fluctuation in flow or influent water quality.

2.3.6 Biological Mechanisms of Removal in Slow Sand Filtration

The main function of biomass in the slow sand filter is to remove and destroy pathogenic microorganisms and viruses, and to facilitate the breakdown of organic matter (Ellis, 1985). Biological treatment mechanisms are most important for particles less than 2 μm in size (Weber-Shirk and Dick, 1997a).

A summary of the biological treatment mechanisms is given by Haarhoff and Cleasby (1991). These include predation of algae and bacteria, scavenging of detritus by aquatic worms found

mainly in the deeper regions of the bed, natural death, inactivation, metabolic breakdown (ie. reduction of organic carbon), and adsorption to the sticky zoogleal surface of the sand. Weber-Shirk and Dick (1997a) suggest that bacterivory or predation of bacteria is the most important of all these mechanisms, and adsorption is the least significant. However, at lower water temperatures, it is suggested that adsorption to biomass is the dominating mechanism, due to reduced biological activity (Welté and Montiel, 1996).

Generally, organisms that inhabit biological processes can either be carnivores, herbivores, bacterivores, detritivores, or omnivores. Duncan (1988) provides a survey of the common organisms that can be found in the sand bed. These include aerobic bacteria, flagellates, ciliates, rotifers, flatworms (microturbellaria), gastrotrichs, nematoda (round worms), anellida (segmented worms), and arthropoda (harpacticoids).

Of all these, the predominant organisms are gram-negative pigmented bacteria such as *Pseudomonas* and *Aeromonas*, as well as algae, protozoa, and higher order eucaryotes (Eighmy et al., 1993). Bacteria that are typically present in biological processes are generally classified as oligotrophs (Rittman and Huck, 1989). Oligotrophs are "characterized by their ability to simultaneously and efficiently utilize a wide array of substrates present at low concentrations" (Moll and Summers, 1996).

The larger microorganisms such as protozoa either feed on suspended particles or bacteria, or are predators of other inhabitants of the sand bed. This is confirmed by Weber-Shirk and Dick (1999) who state, "predators that graze on attached bacteria potentially free up sites for future bacteria attachment while suspension feeding predators directly remove particles from the mobile phase". A proven species to be implicated as a bacterial predator is Chrysophyte (Weber-Shirk, 2002). Other predacious fauna include meiofaunal species (0.1 to 1 mm in size), which feed on individual bacterial or algal cells, suspended particles, or other species (Duncan, 1988). Some eucaryotes are known to be predators to bacteria, while some microorganisms simply produce substances that are toxic to enteric bacteria (Lloyd, 1973; Huisman and Wood, 1974).

Aerobic oligotrophic bacteria grow on the sand media to form a dense biofilm. This sticky biofilm, sometimes referred to as zoogloea, is known to adsorb colloidal material. Some researchers have postulated that filtration efficiency is partially a function of particle adsorption to the sticky biofilm (Huisman et al., 1974). Bacteria such as *Pseudomonas aeruginosa* are known to produce extra-cellular polymeric substances (EPS), polysaccharides and proteins, which serve to anchor bacteria to surfaces (Dai et al., 2002). Bellamy et al. (1985b) suggest that these polymers act to flocculate organisms and destabilize clay and bacteria to facilitate attachment (Galvis et al., 1998). Wheeler et al. (1988) suggests that these extracellular polymers can also provide binding sites for viruses.

Removal of viruses is achieved through microbial predation and adsorption to biomass (Wheeler et al., 1988). Due to the relatively small size of viruses, physical mechanisms of removal are of less importance. Wheeler at al. (1988) found that biomass concentration is just as important for the removal of viruses (eg. rotavirus) as it is for the removal of pathogenic bacteria. In fact, they found similar patterns of removal between viruses and bacteria with respect to depth in the filter.

The term 'bioantagonism' has been used by a few authors to explain a mechanism of removal whereby incoming pathogenic bacteria are either 'out-competed' or 'inactivated' by autochthonous (naturally occurring) bacteria in the sand bed. For example, in the natural environment, Sattar et al. (1999) found that survival of *Cryptosporidium* declined in the presence of autochthonous microorganisms, and this phenomenon was referred to as bioantagonism. Although no specific microorganism was determined responsible for oocyst decay and the actual mechanisms of bioantagonism were unclear, autochthonous bacteria could similarly be responsible for oocyst decay in slow sand filters. This assumption is supported by the research of Uhl (2000), which indicates that the number of pathogens in biofilters decreases rapidly in the presence of autochthonous bacteria. The reasoning is that pathogenic bacteria, or allochthonous bacteria, are accustomed to high concentrations of organic matter where they thrive and experience a high growth rate. However, at low concentrations of organic matter, their growth rate is low. In contrast, the growth rate of

autochthonous bacteria is still high even at low concentrations of organic matter (less than 1 mg/L of carbon), thus out-competing pathogens (Uhl, 2000).

The term, 'inactivation', is used to describe the removal of enteric microorganisms due to predation or bioantagonism (Datta and Chaudhuri, 1991). Each layer of the sand bed has its own inactivation potential (Datta and Chaudhuri, 1991), depending on the vertical distribution of biomass. For example, procaryotes and eucaryotes were active throughout the filter bed in inactivating enteric microorganisms (*E. coli*), however inactivation potential was highest near the surface of filter bed (Datta and Chaudhuri, 1991).

2.3.6.1 Vertical Distribution of Biomass

The vertical distribution of biomass throughout the depth of the slow sand filter is mainly dependent on 'food' availability. The availability of food, or substrate, used for sustenance of the biomass, is dependent on the concentration of influent organic matter and the filtration rate.

For example, in the presence of higher influent turbidity, Datta and Chaudhuri (1991) found that inactivation potential was increased at lower depths of the filter. This was attributed to higher concentrations of organic matter in the influent, resulting in deeper penetration of organics into the sand bed. However, too much organic content in the influent can lead to excessive growth of biomass and clogging of the filter (Cleasby, 1991).

The filtration rate governs the rate at which substrate is delivered to the biomass. Ellis (1985) found deeper penetration of biomass, such as protozoa, at higher filtration rates. In fact, depending on the filtration rate, biomass could be present to a depth of 400 mm or more (Ellis, 1985). Haarhoff and Cleasby (1991) suggest that higher filtration rates would tend to flush algae and detritus particles deeper into the sand bed, thus motivating the migration of protozoa to deeper regions where food is available. Huisman (1977) suggests that because higher filtration rates allow microorganisms to live deeper in the bed, there is an increased risk of bacteria breakthrough.

Eighmy and Collins (1988) analyzed sand bed cores at the end of a filter run and examined the vertical distribution of biomass using the acriflavine direct cell count (AFDC) method and spread plate method. They found very high levels of AFDC and spread plate counts (10^6 to 10^8/g dry wt.) in the schmutzdecke and rapidly declining levels directly below the schmutzdecke, which were about three orders of magnitude lower.

In another study, high densities of protozoa were found as deep as 200 mm (Ellis, 1985). At this depth, amoeba and flagellates were found in densities of 21,600/cm^3 and 64,000/cm^3, respectively (Ellis, 1985).

The vertical distribution of algal species was examined by Bellinger (1979). Bellinger (1979) found motile diatoms with densities of several thousand cells per cm^3 in the top two centimetres of sand and an exponential decrease in densities with depths down to 10 cm, where densities of approximately 100 cells per cm^3 were present. The needle shaped diatoms, bluegreen filaments, and sessile algae penetrated deeper into the sand than the larger and filamentous diatoms. Overall, most of the algae were concentrated in the upper layer of the sand bed.

In another study, Datta and Chaudhuri (1991) examined catalase activity throughout a slow sand filter. Catalase activity was highest at top of filter (0-10 or 0-25 cm). Below the top layer of sand, catalase activity dropped by a factor of 5-10. The varying levels of catalase activity throughout the filter correlated to varying levels of inactivation potential in the sand layers. An interesting finding in this study was that although the inactivation potential below a depth of 25 cm was less than 20% of that of the top layer, the entire bed was still recognized to be active in inactivation.

Temperature can have a profound effect on the vertical distribution of biomass populations. Generally, biological activity is increased at higher water temperatures. Thus, at high water temperatures, Ellis (1985) found larger populations of biomass in the deeper regions of the filter.

The size of the sand media can also influence the vertical distribution of biomass. Basically, pore space increases as media diameter increases. Larger pore space would allow deeper penetration of organic matter into the sand bed, thus providing more availability of food in the deeper regions of the filter.

Overall, the concentration of biomass is greatest at the top of the sand bed where food is most available. Although biomass concentrations decline sharply with depth, the biomass activity throughout the entire filter is important for providing treatment.

2.3.6.2 Dynamic Nature of Biomass

In slow sand filters, maximum treatment efficiency is realized when the filter is fully mature and acclimatized with a steady state population of biomass. A steady state is achieved when equilibrium exists between the microorganism populations and the availability of nutrients (organic carbon) under the ambient water quality conditions. At filter startup, the acclimatization process usually takes 30 to 60 days, or longer during colder water temperatures. Haarhoff and Cleasby (1991) stress the importance of sand bed maturity for optimal treatment efficiency.

In surface water, the ambient water quality conditions may easily fluctuate due to natural and/or anthropogenic influences. Fortunately, biomass populations are consistently dynamic and respond promptly to changes in their environment (Duncan, 1988). As the influent organic concentration, dissolved oxygen, and temperature change, the biomass community will respond by adapting to the given conditions and attempt to re-establish a steady state population.

Fluctuations in temperature, in particular, can have an interesting effect on biomass dynamics in the filter. At increased influent temperatures, Seger and Rothman (1996) found that biomass populations in the top layer of sand were more affected than in deeper layers. This suggests that biomass populations are much more dynamic in the top of the sand bed, where influent water quality conditions are more influential.

2.3.7 Limitations of Slow Sand Filtration

The main drawback of slow sand filtration is its inability to treat high turbidity surface water without the rapid development of headloss and frequent clogging of the filter. Ellis (1985) stated the common belief that "high turbidity in surface waters was the original reason that slow sand filtration had to be rejected in many parts of the United States and led to the development of rapid filtration techniques".

Most surface waters may reach a turbidity of 30-50 NTU, will occasional peaks of 200 NTU after heavy runoff events (Barrett el al., 1991). It is suggested that slow sand filtration operates best with raw water turbidity below 10 NTU, and can manage peaks up to 50 NTU for one or two days without incurring major increases in headloss (Galvis et al., 1992). However, if the turbidity is consistently as high as 50 NTU, the filter will clog and require frequent cleaning, resulting in inadequate filter run times between cleanings. Furthermore, frequent cleaning of the filter disrupts the biological equilibrium in the filter media and does not allow enough time for biological maturation between cleanings (Galvis et al., 1992), leading to increased risk of pathogen breakthrough.

The physical process of clogging in the filter can also have a detrimental effect on effluent quality. The accumulation of particles in the top of the sand bed results in a decrease in pore size and subsequent increase in interstitial water velocity. As the interstitial velocity increases, shearing of the deposited particles increases. This results in the penetration of particles into deeper regions of the filter, and an increased risk of particle breakthrough in the effluent (Huck et al., 2001). In addition, the accumulation of solids can smother bioactive sites in the media, reducing the ability of bacterial predators to prey on harmful bacteria and pathogens (Lloyd, 1974).

Excessive algal matter in the influent (chlorophyll a > 0.05 μg/L, Cleasby (1991)) can also clog filters and require frequent cleaning. High algal concentrations in the influent can also produce taste and odour problems in the effluent. In addition, the presence of excessive algae may increase the pH and cause precipitation of calcium and magnesium in the filter bed,

resulting in an increase in interstitial velocity and decrease in filter efficiency (Galvis et al., 1992).

Slow sand filters are also susceptible to sudden fluctuations in raw water quality, such as sudden increases in solids loading. The biological community in slow sand filters is very dynamic and can adapt to prevailing raw water conditions. However, surface water is prone to sudden fluctuations in water quality due to precipitation events or pollution, thus the biological community in the filter is liable to be upset (Huisman and Wood, 1974). This, in turn, may disrupt the treatment efficiency of the filter.

Another drawback of slow sand filtration is its limited ability to treat stable suspensions of fine colloidal matter. Colloidal matter is much too fine, stable, and numerous to be completely removed by slow sand filtration alone. Colloidal matter is usually derived from clay; it ranges in diameter from 0.3 to 1 μm and exists in suspension due to its negative charge (Montgomery, 1985). Many surface water sources have a high percentage of colloidal matter. For example, an examination of water from a reservoir in Dar es Salaam found 50% of particles less than 1 μm (Boller, 1993). Furthermore, Ingallinella et al. (1998) examined a surface water where 50% of the particles were below 0.5 μm in size.

In slow sand filtration experiments, Bellamy et al. (1985a) found poor removals of turbidity, in the range of 27 to 39%, due to the presence of colloidal clay. However, particles ranging in size from 6.35 to 12.7 μm were effectively reduced by 96.8-98%. Similarly, Fogel et al. (1993) found poor turbidity removals of 55% due to a high amount of colloidal matter in the source water, in which 34.4% of all particles were less than 5 μm in diameter.

The stable behaviour of colloidal suspensions can be even more pronounced in the presence of negatively charged humic matter, which absorb to particle surfaces and prevent particle contact. Ahsan et al. (1996) found that absorbed humic compounds can increase electrostatic repulsion between particles or cause steric hindrance. Indeed, tests have confirmed that attachment efficiency between suspended particles is reduced in the presence of humic matter

(Tipping et al., 1982; Jekel, 1986). Conversely, the presence of Ca^{2+} or Na^{+} in the water can adsorb to the surface of the particle and destabilize a suspension (Collins et al., 1994b).

Slow sand filters are also poor in removing colour. Colour is a surrogate measure of organics and an aesthetic measure of water quality. The dissolved fraction of colour is referred to as true colour. Colour is mainly caused by the presence of organic humic substances (Montgomery, 1985). Due to the stabilizing nature of humic substances on dissolved particles, it is expected that colour is difficult to remove in slow sand filters. Ellis (1985) found that the average expected removal of true colour in slow sand filters is 30%. In fact, the common reason for the declining use of slow sand filtration in the United Kingdom is its limited ability in removing organic colour and dissolved organic carbon (Lambert and Graham, 1995).

Another important limitation of slow sand filtration is its reduced treatment efficiency at low water temperatures. Huisman and Wood (1974), Schuler et al. (1988), Bellamy et al. (1985c), Burman (1962), and Fogel et al. (1993) all report decreased removal efficiencies at lower temperatures, due to decreased microbiological activity and biological treatment. For example, Huisman and Wood (1974) found that the removal of *Escherichia coli (E.Coli)* reduced from 99% to 50% when the temperature decreased from 20°C to 2°C. Ellis (1985) found similar deficiencies in removal, and attributed it to lower predation activity of protozoa at lower temperatures.

Furthermore, Burman (1962) found 99% removal of *E. Coli* and coliform bacteria throughout most of the year in a northern climate, but during persistent cold weather removals decreased to 41% and 88%, respectively. Similarly, Bellamy et al. (1985c) found that removal of total coliform bacteria reduced from 97% at 17°C to 87% at 5 °C, and removal of standard plate count bacteria reduced from 99.9% at 17 °C to 90% at 2 °C. On the other hand, despite a low temperature of 5°C, Poynter and Slade (1977) still found 99.6% and 99.5% removals of coliform bacteria and E. Coli, respectively. The variation in results from these authors could be site specific. Perhaps bacteria removals are less affected by temperature in water that is higher in organic content, resulting in higher biomass levels.

Inadequate nutrient loadings, such as dissolved oxygen and organics, can also limit the performance of slow sand filtration. Aerobic microorganisms in the biomass require an adequate supply of oxygen, carbon, nitrogen, and phosphate for metabolic activity and growth. It is recommended that aerobic biological activity can only be sustained with a minimum dissolved oxygen level of 0.5 mg/L (Visscher et al., 1987), however it should ideally not drop below 3 mg/L (Ellis, 1985).

Finally, from an engineering perspective, a disadvantage of slow sand filtration is its low hydraulic loading rates, forcing the requirements for larger filter areas and higher capital construction costs.

2.3.8 Operational Factors Affecting Removal in Slow Sand Filtration

Slow sand filtration is proven to achieve excellent removals of pathogenic bacteria, protozoa, viruses, suspended solids, and turbidity. However, removal efficiency is highly dependent on the physical and operational characteristics of the filter including the media size, bed depth, filtration rate, biological maturity of the filter, and cleaning practices.

Generally, there are similarities in the findings of many authors, who report a decrease in filter efficiency with increased media size, increased filtration rate, decreased bed depth, and decreased biological maturity of the sand bed.

A smaller media is favoured due to its increased filtration efficiency. Ellis (1985) reports improved bacteria removals with smaller media. Although, the impact of media size on filter performance largely depends on the size distribution and surface chemistry of the particulate matter in the source water. For example, if there are a high proportion of solids in the water with a relatively large particle diameter, they are more likely to be removed, even in larger media. On the other hand, a high proportion of smaller size particles possessing a negative surface charge are more difficult to remove, especially in larger media.

Van der Hoek et al. (1996) documents a varied response from several authors regarding the effect of media size on slow sand filter performance. Interestingly, Bellamy et al. (1985c) reported that an increase in effective sand size did not necessarily result in poor filter performance. An increase in effective media diameter from 0.128 mm to 0.615 mm resulted in only a small decrease in bacteria removals from 99.4% to 96%. Likewise, Van der Hoek et al. (1996) found that the use of a smaller range of grain size (d_{10}=0.19) resulted in only slightly better filtrate quality than a larger range of grain size (d_{10}=0.25). More importantly, however, a shorter filter run length was observed with the smaller media.

Although a larger media may be desired for longer filter runs, it also allows deeper penetration of schmutzdecke and more sand would need to be removed during cleaning to restore headloss to initial values (Bellamy et al., 1985c). This may significantly reduce treatment efficiency after cleaning, leading to an increased risk of pathogen breakthrough.

For example, Burman (1962) found that cleaning of the slow sand filter lead to a reduction in the removal of *E. Coli* from 99 to 94%, although removal of coliform bacteria was unaffected. Burman (1962) also found that removal of chlorine resistant spore-forming bacilli ranged from 81 to 88%, and after cleaning these removals dropped from 81 to 73%. Bellamy et al. (1985a) found that cleaning or replacing the sand resulted in a 1 log decrease in bacteria removal efficiency.

Thus, a good compromise is to use a larger effective media diameter, but preserve filtration efficiency by using a media with a relatively low uniformity coefficient (UC). A lower UC ensures that there is not a high proportion of sand that is too fine, which would lead to premature clogging, and not a high proportion of sand that is too large, which would reduce filtration efficiency (Goitom, 1990).

Filtration rate is another important factor affecting removal in slow sand filters. In particular, sedimentation and biological mechanisms are dependent on filtration rate (Ellis, 1985). This is because a lower filtration allows less turbulent conditions in the sand interstices and

facilitates gravitational sedimentation, reduces fluid shear on deposited particles, and increases the hydraulic retention time in biologically active regions of the filter.

As expected, Poynter and Slade (1977) found that removal of viruses decreased with increased filtration rate. In addition, Muhammad et al. (1996) found that colour removals, which depend mostly on sedimentation, were significantly decreased at higher filtration rates. This confirms that biological treatment and sedimentation are indeed influenced by filtration rate.

Interestingly, Huisman (1977) reported that a higher filtration rate increases the organic loading rate, which results in higher substrate availability and forces microorganisms to live deeper than 300-400 mm in the sand bed, leading to potential breakthrough of bacteria. In some cases, however, filtration rate does not have an effect on bacteria removals. For example, Poynter and Slade (1977) found that increasing the filtration rate from 0.2 m/h to 0.4 m/h had no effect on removals of coliform bacteria and *E. Coli*.

Bed depth is also an important parameter for slow sand filter performance. The minimum depth for good turbidity and coliform bacteria removal is 300 mm, but 600 mm is necessary for removal of all viruses, and perhaps to complete the oxidation of ammonia (Ellis, 1985). Likewise, the removal of colour can be significantly increased at bed depths greater than 400 mm (Muhammad et al., 1996).

Bellamy et al. (1985c) found good removals of bacteria with reduced bed depth. In this study, coliform removals dropped from 97% to only 95% by reducing the bed depth from 0.97 m to 0.48 m. This is because most of the biomass and biological treatment occurs in the upper portion of the sand bed. In fact, Williams (1987) found that all bacteria reduction occurs in the top 20 cm of the filter bed. In this study, a 1 log removal of fecal coliforms was achieved after 5 cm depth and another 1.3 log removal after 20 cm depth, for a total of 2.3 log removal (99.5%).

Overall, bed depth is more important for removal of smaller particles, including viruses, colloidal matter, and colour; and less significant for removal of bacteria.

The biological maturity of the filter also has an important influence on removal efficiency. Basically, if the length of filter run is short and cleaning is frequent, the biological layer will never have enough to time to re-establish equilibrium and maturity. Cleasby et al. (1984b) found that the removal of coliform bacteria increased from 95% to greater than 99% as the filter matured. Likewise, Bellamy et al. (1985a) found that *Giardia* removal was 98% in new sand, where in biologically mature sand, removal was 3 to 4 log. Thus, the importance of lengthy filter runs, which allow plenty of time for maturation, cannot be overstated.

2.3.9 Performance of Slow Sand Filtration

"Slow sand filtration produces an effluent low in turbidity, free of impurities and more importantly, virtually free of bacteria, entero-viruses and protozoa" (Galvis et al., 1988).

Galvis et al. (1998) and Galvis et al. (2002) compiled typical removal efficiencies for slow sand filters from the work of several authors such as Bellamy et al. (1985c), Ellis (1985), Huck (1987), Rachwal et al. (1988), Haarhoff and Cleasby (1991), Hrubec et al. (1991), and Fox et al. (1994). These removal efficiencies are shown in Table 2.2. The original figure was amended to include the results from the work of Cleasby et al. (1984b) and Huisman and Wood (1974). Most of the results are from slow sand filters operating at temperatures above 5^{o}C, filtration rates between 0.04 and 0.2 m/h, bed depths above 0.5 m, and effective media diameters between 0.15 and 0.3 mm.

Table 2.2: Typical Removal Efficiencies for Slow Sand Filtration

Parameter	Effluent or Removal Efficiency	Comments
Turbidity	< 1 NTU	Treatment efficiency depends on quantity, nature, and distribution of particles.
Coliform bacteria	> 99%	Treatment efficiency mostly depends on the biological maturity of the filter.
Entero bacteria	90 to 99.9%	Treatment efficiency affected by temperature, filtration rate, media size, bed depth, and cleaning practices.
Entero viruses and *Giardia*	99 to 99.99%	Effect of cleaning practices on removal efficiency in a biologically mature bed is minimal.
True colour	25 to 40%	Colour is associated with organic material and humic acids. Average 30% removal.
Total organic carbon (TOC)	< 15 - 25%	
Dissolved organic carbon (DOC)	5 - 40%	Mean 16%
Biodegradeable dissolved organic carbon (BDOC)	46 - 75%	Mean 60%
Assimilable organic carbon (AOC)	14 - 40%	Mean 26%
UV absorbance (254 nm)	5 - 35%	Mean 16-18%
Trihalomethane (THM) precursors	< 25%	
Iron and manganese	30 to 90%	Fe levels > 1 mg/L reduce filter run length due to precipitation and filter clogging.

Adopted from Galvis et al. (1998/2002)

2.3.9.1 Removal of Bacteria

It is suggested that slow sand filtration can achieve between 99 and 99.9% of pathogenic bacteria (Van Dijk and Ooman, 1978). However, removal efficiencies may be somewhat site specific as there is some variation in the findings from several authors. The variation in bacteria removals can be attributed to differences in source water quality conditions (as discussed in section 2.3.7) and filter operational conditions (as discussed in section 2.3.8). This highlights the importance of onsite pilot testing to determine treatment performance under the prevailing water quality and operational conditions.

2.3.9.2 Removal of Viruses

Slow sand filtration can achieve very good removals of viruses. Typical virus removals in slow sand filtration range from 2 to 6 logs (Troyan and Hansen, 1989), and generally increase

with increasing bed depth and decreasing filtration rate (as discussed in section 2.3.8), and increasing water temperature.

Poynter and Slade (1977) found 99.9% removal of poliovirus 1 with a bed depth of 600 mm and filtration rate of 0.2 m/h. Removal efficiencies decreased with lower bed depth and higher filtration rates, and were only slightly affected by temperature. For example, 99.999% removal was achieved at a temperature of 11 to 12 ^{o}C, but decreased only slightly to 99.8% at 6 ^{o}C.

Yahya et al. (1993) studied the removal of bacteriophages MS-2 and PRD-1, which represent human enteric viruses because they are similar in shape and size (25 nm and 62 nm, respectively) and they absorb poorly to sand. Removal of MS-2 and PRD-1 was 99% and 99.9%, respectively.

2.3.9.3 Removal of Giardia and Cryptosporidium

Slow sand filtration is very efficient in removing *Giardia* and *Cryptosporidium*. A summary of removals reported by several authors is presented in Table 2.3.

In general, *Cryptosporidium* is more difficult to remove than *Giardia* because, due to its smaller size, it has a lower collector efficiency than *Giardia* (Hsu et al., 2001).

Table 2.3: Removals of *Giardia* and *Cryptosporidium* in Slow Sand Filters

Author	Giardia	Cryptosporidium	Comments
Bellamy et al. (1985a)	> 98%		
Schuler et al. (1988)	99.83 to 100%	100%	
Schuler et al. (1991)		3.9 to 7.1 log	
Fogel et al. (1993)	average of 93%		
Logsdon et al. (1993)	93.7 to 99.99%		
USEPA (2001b)		> 3.7 log	
Logan et al. (2001)		99.9 to 99.99%	Influent spike of 65,000 oocyst/L, Less removal with larger media
Timms et al. (1995)		99.997%	Influent spike of 4,000 oocyst/L, Filtration rate of 0.3 to 0.4 m/h

Bellamy et al. (1985a) and Bellamy et al. (1985b) found that *Giardia* cyst removal in slow sand filters was virtually 100% and was not greatly influenced by the filtration rate, media size, or bed depth. *Giardia* removal, however, was greatly influenced by the biological maturity of the filter where removal in new sand was 98%, and removal in biologically mature sand was 3 to 4 log (Bellamy et al., 1985a/b). In addition, once the filter bed is biologically mature, Bellamy et al. (1985a) found that disturbance of the schmutzdecke, such as during cleaning of the filter, does not significantly affect cyst removal.

In an interesting study, Timms et al. (1995) spiked the influent of a slow sand filter with 4,000 oocysts/L, and found removals of greater than 99.997%. More importantly, sand samples from the filter were analyzed for oocysts and all of the oocysts recovered were found in the upper 2.5 cm of the filter. It was suggested that with prolonged filtration, oocysts may eventually penetrate deeper in the bed, but it is unknown whether they would be viable or not.

Cryptosporidium oocysts are much smaller than the pores in the media, however Timms et al. (1995) recovered all of the oocysts in the top 2.5 cm of the sand bed. Since adequate space for oocyst transport exists, processes other than physical straining mechanisms are responsible for removal (Logan et al., 2001).

Logan et al. (2001) conducted an interesting experiment where slow sand filters were spiked with 65,000 oocysts/L and removals of oocysts were 3 to 4 log. More importantly, it was found that biofilm production likely resulted in decreased effective pore size and increased efficiency of "collection processes dependent on hydrophobic or electrostatic interactions" as described by DLVO theory.

On the other hand, in smaller media, the decrease of effective pore size actually decreased removals as "channelling and preferential flow" occurred due to increased interstitial velocity, leading to particle breakthrough. These findings show that, besides DLVO mechanisms, media size can have a significant effect on oocyst removal. In addition, the effect of filtration rate is more significant in larger media (Logan et al., 2001).

The uniformity coefficient (UC) of the media can also have a significant effect on the removal of *Cryptosporidium* in slow sand filters. Schuler et al. (1988) found *Cryptosporidium* removals of 100% with a UC of 1.67, while Fogel et al. (1993) found removals of 48% with a UC of 3.5-3.8, which exceeds the maximum UC of 3, as recommended by Huisman and Wood (1974). Thus, the increased media sizes (d_{60}=1.14 mm) and pore sizes associated with a UC of 3.8 resulted in less capture of organic substrate for growth and metabolism of the sticky zoogleal biofilm (Fogel et al., 1993). This resulted in decreased physical capture and biological degradation of particles, ultimately allowing oocysts to pass through the bed without inactivation.

The pH level can also have a significant impact on the removal of oocysts and cysts in the sand bed. As pH increases, ionization of amino acids on the surface of the oocyst occurs (Hsu et al., 2001). This increases the repulsive force between the oocysts and media, decreases collision efficiency, and lowers removal rates.

In addition, the removal efficiency of *Giardia* and *Cryptosporidium* is lower at decreased temperatures (Fogel et al., 1993), due to decreased biological activity. For example, Schuler et al. (1991) found that Giardia removals were only 2 to 3 log during winter months, compared to 4 log in warmer months. This represents a potential health risk during the winter months of northern climates.

While it is proven that biological degradation is an important factor in the removal *Cryptosporidium*, the physical shear of oocysts in filters can also affect their viability in the sand bed (USEPA, 2001a).

Overall, although slow sand filtration can achieve excellent removals of *Giardia* and *Cryptosporidium*, there are many factors that can reduce treatment efficacy, potentially resulting in the breakthrough of pathogens in the effluent. Thus, the USEPA (2001b) recommends a multi-barrier treatment approach to provide drinking water that is consistently safe for human consumption.

2.3.9.4 Removals Recognised by the United States SWTR

It is important to understand the current accepted standard of removal as stipulated in the U.S. Surface Water Treatment Rule (SWTR). The SWTR stipulates a requirement for 99% removal of *Cryptosporidium*, 99.9% removal of *Giardia* and 99.99% removal of viruses of fecal origin. In addition, the international standard of effluent turbidity was lowered from 1 NTU to 0.5 NTU in 1993 (USEPA, 1999).

In the SWTR, slow sand filtration is considered capable of achieving at least 2 log removal of *Giardia* (99%) and at least 1 log (90%) of viruses (Logsdon, 1987b; USEPA, 1988b). Although studies have shown that slow sand filtration is capable of greater than 3 log removal of *Giardia* (Logsdon, 1987b) and 3 log removal of viruses (Poynter and Slade, 1977), filter designers should assume a 2 log removal for *Giardia* and 1 log removal for viruses, and conservatively allow for sufficient subsequent disinfection to provide the rest of the removal.

The SWTR identifies critical factors that can affect removal efficiencies. These include raw water turbidity greater than 1 NTU, cold water temperatures, and poor operating conditions (non-constant flow rate, intermittent filter operation, etc.) (USEPA, 1991). In addition, an effective media size of 0.15 to 0.3 mm is recommended. Cleaning of the sand bed should occur when the headloss through the bed reaches 1 to 1.5 m. After cleaning, the operator should allow for a sufficient ripening time to re-establish the biological layer. Typically, the filter effluent should not be used for drinking water until the turbidity returns to less than 1 NTU (USEPA, 1991).

Furthermore, the SWTR stipulates certain performance requirements to ensure successful operation of a slow sand filter. For example, turbidity measurements should be less than 1 NTU in 95% of the measurements each month, and the effluent turbidity should never exceed 5 NTU.

Overall, if the above operational, water quality, and performance requirements are met, an interim credit of up to 3 log removal of *Giardia* will be formally recognized for the given slow sand filtration system.

2.3.9.5 Surrogate Measurements of Giardia and Cryptosporidium

Nieminski and Ongerth (1995) found a high correlation between the removal of cysts and oocysts and the removal of respective size particles (r^2=0.82 and r^2=0.79, respectively). There was a slightly lower correlation between the removal of cysts and oocysts and the removal of turbidity (r^2=0.64). This suggests that the "log removal of turbidity can serve as a rough indicator of treatment performance for cyst and oocyst removal, but with lower accuracy than particle counting" (Nieminski and Ongerth, 1995).

Similarly, LeChevallier and Norton (1992) found that removal of particles greater than 5 μm and turbidity were useful indicators of cyst or oocyst removal. For example, a 1 log removal of particles and turbidity corresponded to a 0.66 and 0.89 log removal of cysts and oocysts, respectively.

However, these findings conflict with the work of Schuler and Ghosh (1991), who observed removals of *Cryptosporidium* and *Giardia* exceeding 99.99%, however the removal of similar sized particles was much less (about 40%) than the removal of the actual organisms. It is suggested that biological degradation is more influential in the removal of oocysts than like-sized particles. Thus, particle counting might not be a viable method for determining the actual removal of *Cryptosporidium* and *Giardia*.

2.4 ADVANCES IN SLOW SAND FILTRATION

2.4.1 Removal of Natural Organic Matter

Slow sand filtration has always been viewed as a biological treatment process that serves mainly to remove pathogenic bacteria, protozoa, and viruses. In the past 20 years, however, there has been a returning interest in biological treatment processes to remove natural organic matter (NOM) in surface waters.

NOM is ubiquitous in surface water and is generally in the form of suspended, dissolved, and colloidal fractions. Colloidal and dissolved organic matter (DOM) in natural waters consists of 40-80% humic substances (Müller et al., 2004), namely humic and fulvic acids.

Humic substances arise from the decomposition of plant and animal materials (Stephenson et al., 1979). It is common in surface waters, especially in northern climates (Melin and Odegaard, 2000). Generally, humic substances cause taste, odour, and colour problems in drinking water and are therefore aesthetically undesirable by consumers.

More importantly, humic substances or DOM poses a concern for water treatment practitioners as the oxidation of DOM with chlorine and/or other oxidants can form harmful disinfection by-products (DBPs), such as trihalomethane (THM). Moreover, high levels of DOM represent a potential for bacterial growth in water distribution systems.

Biodegradable organic matter (BOM) is defined as the portion of DOM that can be oxidized by heterotrophic organisms (Carlson and Amy, 1995) in biological treatment processes. It has been found that the removal of BOM reduces the required disinfectant dosage (chlorine demand) and residual, which results in a decrease in the formation of DBPs by removing precursor material (Huck et al., 1998). In addition, removal of BOM reduces bacterial regrowth in distribution systems.

The mechanism by which organic matter is reduced in biological filters is complex. Heterotrophic bacteria utilize protein and carbohydrates that are present in the NOM, as well as aromatic and aliphatic compounds (Eighmy et al., 1993). As easily biodegradable low molecular weight organic matter constitutes less than 20% of the total organic matter in surface waters, heterotrophic microorganisms produce extra-cellular enzymes to hydrolyse macromolecular organic matter to release low molecular weight organic compounds (Hendel et al., 2001).

Typically, slow sand filtration is capable of removing 5-25% of NOM (McMeen and Benjamin, 1996), 15-25% of TOC (Galvis et al., 1998), and 50% of BOM (Ellis, 1985).

2.4.1.1 Measurement of NOM and BOM

Ultraviolet (UV) absorbance at a wavelength of 254 nm and dissolved organic carbon (DOC) are generally considered to be acceptable surrogates for the measurement of NOM (McMeen and Benjamin, 1996).

UV absorbance of natural waters at a wavelength of 254 nm is attributed to the presence and amount of aromatic structures (Korshin et al., 1997). Specific ultraviolet absorbance (SUVA) represents the aromatic content of the organic carbon and is defined as the ratio of UV absorbance to the DOC. A high value of SUVA (≥4 L/mg-m) indicates a more hydrophobic, aromatic, and higher molecular weight organic matter. A low value of SUVA (≤3 L/mg-m) indicates a more non-humic, hydrophilic and lower molecular weight matter (White, 1997; Edzwald & Van Benschoten, 1990). Generally, higher molecular weight organic matter is more difficult to biodegrade than lower molecular weight organic matter. This is consistent with the work of Barrett and Silverstein (1988) who found that TOC removal was compromised with higher proportions of humic compounds in the source water.

Most importantly, it was also found that higher SUVA values represent a higher DBP formation potential, mainly due to the finding that disinfectants primarily oxidize sites on the aromatic structures (Korshin et al., 1997) in the humic matter. Thus, SUVA is an important parameter for indicating the DBP formation potential of a given source water.

Total organic carbon (TOC) is a measure of the total organic carbon content in the water including the suspended and dissolved fractions. Dissolved organic carbon (DOC) is the fraction of the TOC that passes through a 0.45 μm filter. It has been found that DOC represents 83 to 98% of the TOC content (Owen et al., 1993). DOC is made up of biodegradable dissolved organic carbon (BDOC) and non-purgeable dissolved organic carbon (DPDOC). Eighmy et al. (1993) reports findings from several authors, which have found that the biodegradable fraction of organic matter ranges from 5 to 60% of the non-purgeable dissolved organic carbon (NPDOC).

Measurement of BOM is commonly achieved by measuring assimilable organic carbon (AOC) or biodegradable dissolved organic carbon (BDOC). AOC tests "measure BOM as the increase in biomass as assayed by plate counts, ATP, or other biomass-specific parameters" (Woolschlager and Rittman, 1995). BDOC tests measure BOM as the biodegradable fraction of the dissolved organic carbon. Alternatively, a simple tool for estimating BOM removal is the measurement of dissolved oxygen (Huck et al., 2000).

2.4.1.2 Factors Affecting the Removal of BOM

Carlson and Amy (1998) found that the removal of DOC during biofiltration, referred to as $BDOC_{filter}$, was limited by the biomass concentration in the filter bed. Also, $BDOC_{filter}$ remained constant when the hydraulic loading rates were decreased. On the other hand, removal of DOC was decreased at higher than normal loading rates, suggesting that the available biomass that had acclimatized to the given hydraulic loading conditions could not assimilate the higher availability of carbon resulting from the sudden increase in BOM loading rate.

It is expected, however, that $BDOC_{filter}$ at higher hydraulic loadings would be higher if the biomass were first allowed to acclimatize to the higher BOM loading rate under steady state conditions. These findings indicate the importance of maintaining a steady state hydraulic loading rate throughout the filter, and suggest that biofiltration efficiency is compromised under sudden changes in water quality or increases in hydraulic loading rate.

As expected, Carlson and Amy (1998) found higher biomass concentrations under higher steady state hydraulic loading rates, suggesting that the rate of carbon utilization was higher. However, despite the higher biomass concentration, the $BDOC_{filter}$ was not affected, suggesting that DOC or BOM removal was limited by the availability of BDOC. The availability of BDOC is dependent on the source water, or in this case, the applied ozone dose, where a maximum production of BDOC was achieved at 1 mg O_3/mg DOC (Carlson et al., 1996). Overall, these findings indicate that if filters were allowed to acclimatize under steady state conditions, they would achieve the same removal of DOC or BOM, regardless of the hydraulic loading rate.

Another important parameter affecting the removal of BOM in biofiltration processes is the empty-bed contact time (EBCT). EBCT is the hydraulic detention time of the volume in the filter that is occupied by the media. Carlson and Amy (1998) found that biomass formation with respect to empty-bed contact time (EBCT) was independent of hydraulic loading rate, suggesting that EBCT is a useful parameter for biofilter design. These findings are confirmed by Moll and Summers (1996), who found that BDOC removal and biomass were dependent on EBCT, but independent of hydraulic loading rate.

Although, the use of EBCT as a design parameter for slow sand filters may be less applicable. This is because slow sand filters have much finer media than conventional filters, thus preventing the penetration of large biomass concentrations into the deeper regions of the filter bed. However, the principle that BOM removal is increased with higher EBCT would still hold true in the upper, biologically active region of the sand bed, especially at lower hydraulic loading rates, which would allow a longer retention time in the upper layer. Interestingly, Moll and Summers (1996) found that quickly biodegradable BDOC was fully removed at very low EBCTs, corresponding to the top few centimetres of the biofilter (rapid sand filter in this case). This result provides some reassurance that the concept of EBCT could at least be applied to the upper, biologically active layer of slow sand filters, which have even lower EBCT than rapid sand filters.

The work of Huck et al. (2000) is also important for identifying other critical factors affecting BOM removal in biological processes. In this study, it was found that BOM removal increases with longer contact time and larger L/d ratios, which is especially important for removal of disinfection by-products, chlorine demand, and high levels of BOM. This is expected because longer contact times between the BOM and biomass will result in increased biological treatment. It was also confirmed that removal of easily biodegradable BOM is significantly impaired at water temperatures below 5^{o}C.

2.4.2 Ozone and Slow Sand Filtration

Ozonation prior to slow sand filtration can have a positive influence on slow sand filter treatment performance, microbiology, and NOM transformation (Eighmy et al., 1993). As slow sand filtration alone is only capable of removing 5-25% of NOM (McMeen and Benjamin, 1996), pre-ozonation is important for achieving good removals of NOM from surface water.

Generally, ozone increases the biodegradability of NOM, hence increasing biological activity in the filter and increasing the removal of BOM. It works by oxidizing long chain molecular substances into more easily biodegradable short chain molecular substances. More specifically, Eighmy et al. (1993) found that ozonation of NOM produced low molecular weight substances capable of supporting bacterial growth consisting of obligate aerobic bacteria or facultative anaerobic bacteria with "robust and diverse heterotrophic capacities" Eighmy et al. (1993).

Thus, pre-ozonation increases the BDOC content of NOM, increases removal of UV absorbance, and also transforms NOM into more hydrophilic forms, which have a lesser potential for THM formation (Eighmy et al., 1993). Mogren et al. (1990) studied the ozonation of different source waters, and found that it increased the BDOC fraction of water from 30 to 60%, 10 to 40%, or 0 to 20%, depending on the characteristics of the source water. This clearly indicates that ozonation is effective in increasing the biodegradability of NOM.

Gould et al. (1984) found NPDOC and UV absorbance removals of 15 to 25% and 39 to 54%, respectively, in pre-ozonated slow sand filter effluent. The ozone dose was 3 to 5 mg O_3/L and the slow sand filtration rate was 0.14 m/h. Comparatively, non-ozonated slow sand filter effluent had removals of only 9 to 14% and 11 to 15%, respectively.

Similarly, Zabel (1985) found NPDOC and UV absorbance removals of 35 and 70%, respectively, operating at an ozone dose of 5 mg O_3/L and filtration rate of 0.25 m/h. Whereas, non-ozonated slow sand filters had NPDOC and UV absorbance removals of only 12 and 16%.

Rachwal et al. (1988) found similar trends in the removal of UV absorbance, but more importantly, found that most of the removal occurred during pre-ozonation, as the slow sand filter contributed only 10% removal. This is confirmed by Eighmy et al. (1993), who report that ozonation of NOM reduces UV absorbance of the source water likely through oxidation of aromatic compounds.

Ellis (1985) found that removal of BOM in pre-ozonated slow sand filter effluent was 75%, compared to 50% achieved by slow sand filtration alone. In addition, removal of TOC in pre-ozonated slow sand filter effluent was 35%, compared to 15% achieved by slow sand filtration alone.

The effects of ozonation on TOC reduction are even more profound during the winter, when TOC removal by slow sand filtration alone is generally poor (Seger and Rothman, 1996). In fact, Seger and Rothman (1996) found that reduction of TOC in cold water (<8 ^{o}C) improved 220% with pre-ozonation, compared to an improvement of 75% in warm water (>8 ^{o}C). Thus, pre-ozonation is especially important during cold water conditions for optimal removal of NOM in the slow sand filter.

2.4.2.1 Removal of Colour with Ozonation

Ozonation is also used for the removal of true colour, as slow sand filtration typically only achieves 30% removal. Greaves et al. (1988) found that pre-ozonation removed true colour by 74% followed by an additional 20% removal by slow sand filtration. Comparing ozonated and non-ozonated slow sand filter effluent, Cable and Jones (1996) found a 52% removal of true colour compared to 19% in non-ozonated effluent. In the case of colour removal, ozonation alone is responsible for most of the removal, due to the alteration of the chemical bonds in humic materials, "leading to a reduction in the conjugation of the molecules" (Cable and Jones, 1996).

2.4.2.2 The Effect of Ozone on Filter Run Length

Another advantage of pre-ozonation is extended slow sand filter run times. Rachwal et al. (1988) found that filter run times were extended due to the oxidation of filter blocking algae. This phenomenon is likely more significant in tropical regions or during the warmer seasons of northern regions.

Conversely, Eighmy et al. (1993) found that pre-ozonation actually reduced filter run times, most likely due to increased biomass development in the filter. In this case, a larger media with more biomass storage capacity can be used to extend filter run times.

2.4.3 Removal of Disinfection By-Products with Slow Sand Filtration

2.4.3.1 Removal of Trihalomethanes

Chlorination has been used throughout the 20th century to prevent transmission of waterborne disease. Recently, however, there is a large concern over the formation of chlorination by-products, such as trihalomethanes (THMs), which pose a cancer risk in humans.

THMs are formed when a chlorine-based chemical oxidant reacts with organic matter in water. The USEPA has stipulated a maximum contaminant level (MCL) of 80 μg/L for total THMs (USEPA, 1998a). Filtration is very important for the removal of THM pre-cursor material and the reduction of THM formation potential (THMFP) during the post-chlorination stage.

Eighmy and Collins (1988) found that removal of THMFP in slow sand filtration ranged from 9-27% in the winter and 14-27% in the fall. The improved removals in the fall were attributed to increased growth of biomass in warmer water temperatures. In fact, Eighmy and Collins (1988) found a direct correlation between the amount of biomass concentration and the removal of THMFP. For example, filters cleaned with the biomass conserving harrowing method, as opposed to scraping, had significantly higher removals of THMFP.

In pre-ozonated water, Eighmy et al. (1993) found even better removals of THMFP, 40 to 70%, compared to 10 to 15% removal in conventional systems. They used ozone doses of 2 to 6 mg 0_3/L, and found that the increased THMFP removal was due to both direct oxidation and enhanced biological treatment.

Cable and Jones (1996) also found improved removals of THMFP with pre-ozonation, averaging 50% for ozonated effluent compared to 28% for non-ozonated effluent. They also found good linear correlations between THMFP and true colour or TOC. This suggests that true colour and TOC are reasonable surrogates for predicting THMFP.

Overall, removal of THMFP is enhanced with higher water temperatures and pre-ozonation.

2.4.3.2 Removal of Ozonation By-Products

Although pre-ozonation is very effective in aiding removal of BOM, it may unfortunately result in the formation of DBPs or ozonation by-products (OBPs) that are harmful to human health. Examples of OBPs are low molecular weight aldehydes and organic acids, including formaldehyde and acetaldehyde, as well as bromate. Bromate, a suspected carcinogen, is formed when bromine is present in the water. The concentration of bromate that is formed during ozonation depends on the ozone dose, contact time, and pH (Kimber, 2003). The current MCL of bromate is 10 µg/L (USEPA, 1998a).

Fortunately, biological processes can be very efficient in removing OBPs. In fact, Eighmy et al. (1993) found that formaldehyde and acetaldehyde were readily removed to below detection limits during slow sand filtration.

However, the removal of some ODPs is highly dependent upon EBCT. Melin and Odegaard (2000) found that the removal of ODPs increased with increasing EBCT and the optimal EBCT was around 20 minutes. This contact time could easily be achieved in the biologically active regions of a slow sand filter.

There is very little information available on the removal of bromate by slow sand filtration, but it is expected that increased EBCT will result in optimal removals.

2.4.4 Post-Treatment with Granular Activated Carbon Filtration

Granular activated carbon (GAC) is a common adsorbent used in drinking water treatment for the removal of dissolved organic matter and humic substances, THM precursor material, taste and odour compounds, pesticides, and ozonation by-products.

GAC has a large surface area consisting of macropores and micropores, which have a large adsorptive capacity. It is also capable of supporting a high amount of biomass, which is sheltered from fluid shear forces. One problem with biomass supporting GAC filters, however, is that biomass can detach from the media and be net producers of bacteria (Uhl and Gimbel, 1996).

GAC filters can be operated as a post-slow sand filtration treatment step, just prior to disinfection, or as a layer within the slow sand filter. This adds robustness to the process, as GAC can provide additional removal of true colour, colloidal matter, and THMFP, beyond that which is capable in slow sand filtration alone.

For example, GAC filters operated at slow filtration rates have been reported to remove greater than 90% of organic precursor materials (Fox et al., 1984). More impressively, Eighmy and Collins (1988) found organic precursor removals of greater than 75% in a GAC amended slow sand filter with a GAC depth of only 7.6 cm above the sand bed.

Mallevaille and Duguet (1988) found that GAC filters operated at a filtration rate of 0.625 m/h (EBCT of 14.4 min.) removed an additional 15% of TOC after a 10% removal by slow sand filtration, giving a total TOC removal of 25%.

Dussert and Tramposch (1996) summarized the results from many authors regarding the removal of AOC, ozonation by-products, DOC, TOC, and THMFP in GAC filters, and found removals of 42-57%, 73-90%, 17%, 29%, and 21-40%, respectively.

Thus, post-treatment with GAC filtration can provide a significant amount of additional treatment, beyond that which is capable by slow sand filtration alone. Furthermore, it is an important treatment barrier in the multi-barrier approach to water treatment.

2.5 BACKGROUND ON MULTISTAGE FILTRATION

2.5.1 Addressing the Limitations of Slow Sand Filters with Multistage Filtration

A proven treatment method to cope with many of the limitations of slow sand filtration is multistage filtration (MSF). MSF is a robust, multi-barrier treatment method, which consists of pre-treatment with roughing filtration followed by slow sand filtration. Although, in recent years the addition of pre-ozonation and granular activated carbon (GAC) filtration stages have been practiced. MSF can consistently provide effluent water quality that exceeds the capabilities and limitations of slow sand filtration alone. Furthermore, Galvis et al. (1998) advocates that MSF is suitable for rural communities of small to medium size and remote areas in northern climates (Galvis et al., 1998).

A diagram depicting the multi-barrier treatment concept is shown in Figure 2.2. In the diagram, the surface water undergoes a step-by-step treatment process. In the pre-treatment or roughing stage, the larger sized particles (mainly suspended inorganic matter) are efficiently removed while the smaller particles (pathogens, bacteria, suspended solids, etc.) are gradually reduced. In the main treatment or slow sand filtration stage, the smaller particles are completely removed. Thus, particulate matter and pathogenic microorganisms face a series of treatment barriers throughout the treatment system. As the water becomes progressively cleaner in the direction of flow, it becomes increasingly difficult for pathogens to penetrate through the multi-barrier treatment system.

Thus, a multi-barrier approach to water treatment, such as that provided by multistage filtration is recommended for providing a reliable source of drinking water that is consistently safe for human consumption.

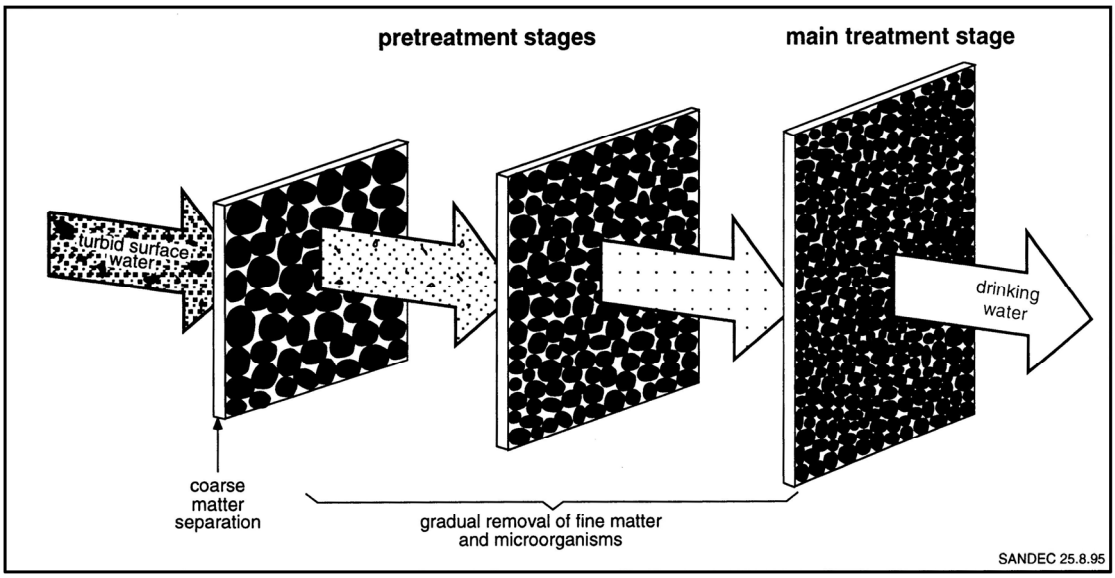

Figure 2.2: Diagram of the Multi-Barrier Treatment Concept

(Reprinted by permission of the Swiss Centre for Development Cooperation in Technology and Management (SKAT), from Surface Water Treatment by Roughing Filters: A Design, Construction, and Operation Manual (Sandec Report No. 2/96), by Wegelin, M., SKAT, 1996).

2.5.2 Benefits of Roughing Filtration

Gravel filtration has been used in water treatment since the early 1800s, when it was first used in Scotland (Baker, 1948) to pre-treat water prior to slow sand filtration. Gravel filtration soon disappeared due to the advent of chemical and mechanical water treatment, but resurfaced in the 1970s and 1980s mainly in developing countries to pre-treat high turbidity water prior to slow sand filtration (Collins et al., 1994b). As roughing filters do not require sophisticated mechanical equipment or the use of coagulants (Wegelin and Schertenleib, 1993), they are a sustainable method of pre-treatment in rural areas.

Briefly, roughing filters consist of several gravel media layers ranging in size from 20 mm down to 4 mm (Wegelin and Schertenleib, 1993), in the direction of flow. There are several types of roughing filters, which are shown in Figure 2.3. Generally, they are classified based on the direction of flow (upflow, downflow, or horizontal flow) and the depth of media layers in the direction of flow. Selection of the roughing filters depends on the raw water characteristics and the operation and maintenance requirements (Galvis et al., 1993). Further description of the different roughing filter types and selection criteria is given in section 2.5.3.

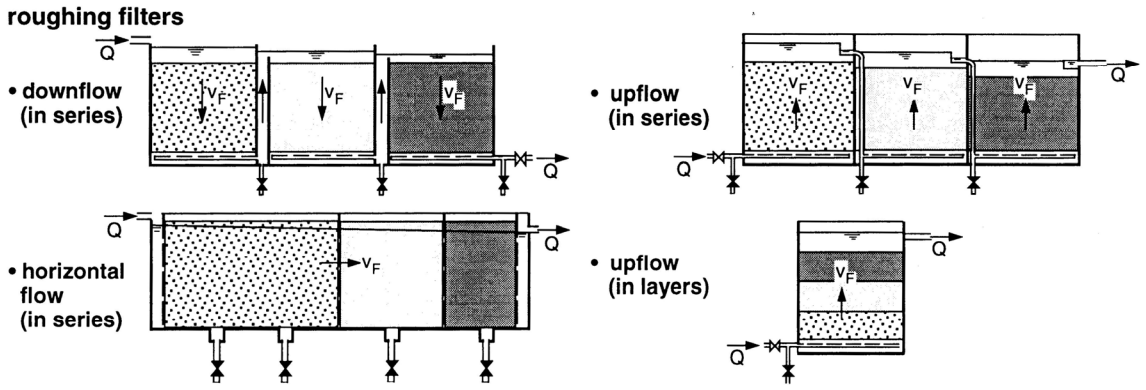

Figure 2.3: Types of roughing filters

(Reprinted by permission of the Swiss Centre for Development Cooperation in Technology and Management (SKAT), from Surface Water Treatment by Roughing Filters: A Design, Construction, and Operation Manual (Sandec Report No. 2/96), by Wegelin, M., SKAT, 1996).

Traditionally, the main function of roughing filtration is to reduce influent turbidity and suspended solids to levels that are suitable for effective operation of slow sand filters. It can also reduce filter-clogging algae, stable suspensions of colloidal matter, and pathogens without the addition of coagulants. Generally, pre-treatment is recommended for raw water turbidities greater than 10 NTU (Bernardo, 1988).

Essentially, the roughing filter protects the slow sand filter from getting overloaded with particulate matter, thus limiting headloss development, and enabling longer filter runs. Actually, it has been found that with pre-roughing filtration, slow sand filters can achieve filter runs that are 5 times longer than without pre-roughing filtration (Wegelin and Schertenleib, 1993). Roughing filters can have filter runs up to a year with raw water that is periodically high in solids loads (Boller, 1993). This is because they are designed to allow deep penetration of solids, resulting in a large capacity for solids storage. They also have low headloss due to their large pores, which are described by Rajapakse and Ives (1990) as lens-like cavities underneath the media grains.

The roughing filter can also attenuate sharp and sudden peaks in turbidity and fluctuations in water quality, thus protecting against the disruption of the biological community in slow sand

filter. In addition, it provides increased retention times and additional surface area for biological treatment, which becomes increasingly important during low water temperatures.

Finally, from an engineering perspective, pre-treatment with roughing filtration can potentially allow the slow sand filter to be operated at higher hydraulic loadings, thus reducing the filtration area and lowering construction costs. Past experience in Zurich and London has shown that, with adequate pre-treatment, slow sand filtration can operate effectively at filtration rates as high as 0.4 to 0.8 m/h (Galvis et al., 1998).

Overall, multistage filtration, which involves pre-treatment with roughing filtration prior to slow sand filtration, is not only important for successful operation of the slow sand filter, but is also a robust approach to improving water quality to consistently safe levels for human consumption.

2.5.3 Types of Roughing Filters

A horizontal roughing filter (HRF) consists of a horizontal filter box with 3 or 4 compartments of decreasing length separated by baffles, in which water flows horizontally. Each compartment is filled with gravel, with the coarsest media in the first compartment and the finest media in the last compartment. The advantage of HRF are its extended bed lengths and solids storage capacity, resulting in less cleaning frequency than upflow roughing filters (Collins et al., 1994a). It is also more suitable for treating very high suspended solids concentrations. The disadvantage of the HRF is its large space requirements. A diagram of a HRF and its typical design parameters are shown in Figure 2.4.

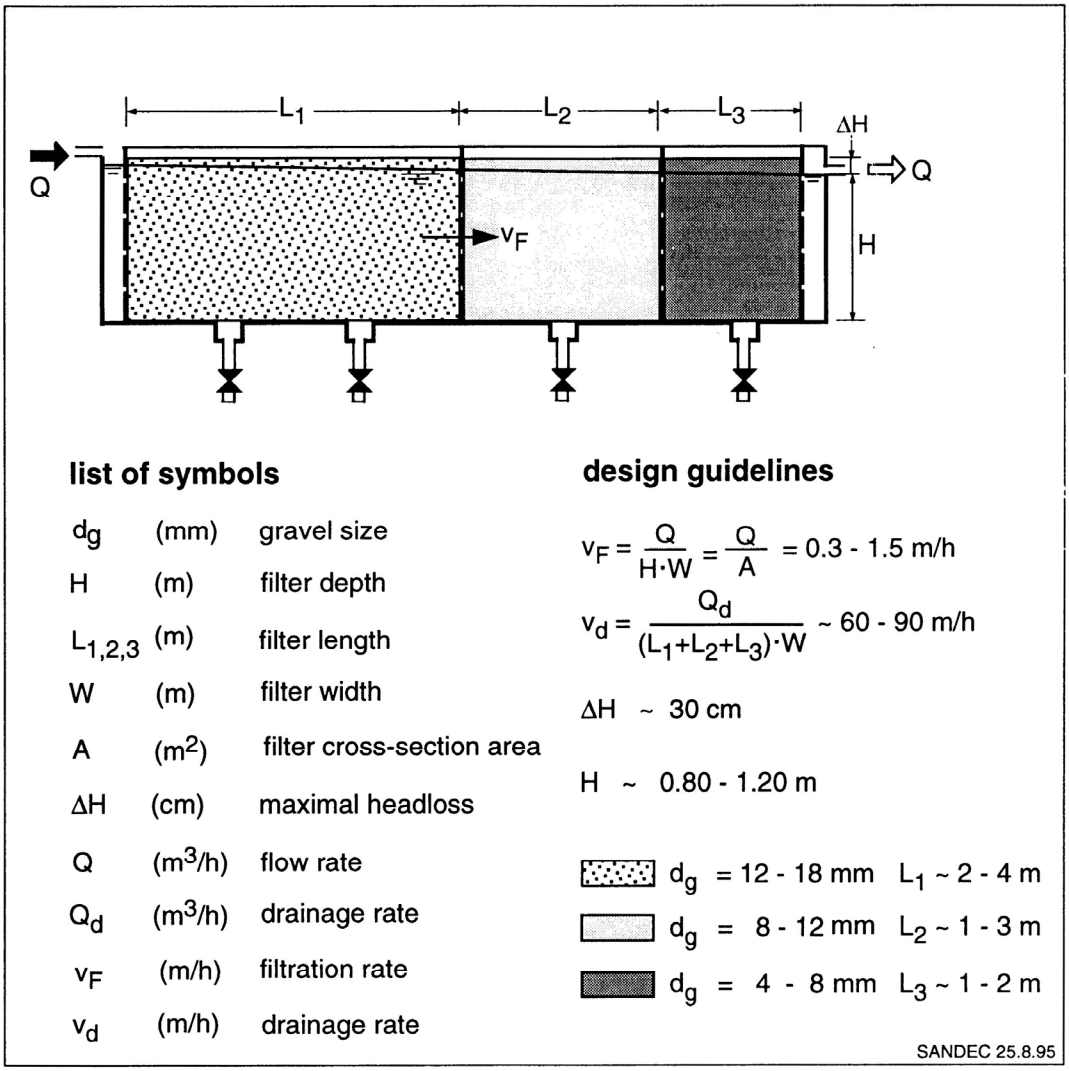

Figure 2.4: Diagram of a HRF and Typical Design Parameters

(Reprinted by permission of the Swiss Centre for Development Cooperation in Technology and Management (SKAT), from Surface Water Treatment by Roughing Filters: A Design, Construction, and Operation Manual (Sandec Report No. 2/96), by Wegelin, M., SKAT, 1996).

A downflow roughing filter in series (DRFS) consists of 3 or 4 individual filter boxes, each filled with gravel, with the coarsest media in the first compartment and the finest media in the last compartment. Water flows downward through each media compartment. A diagram of the DRFS and its design parameters are shown in Figure 2.5.

An upflow roughing filter in series (URFS) is similar to the DRFS except that water flows upward through each media compartment. As most of the solids accumulation occurs in the

bottom of the filter near the drainage pipes, cleaning is much more efficient in upflow roughing filters. Thus, although upflow and downflow roughing filters perform similarly (Wegelin, 1996), upflow roughing filters are recommended for ease of cleaning. A diagram of the URFS and its design parameters are shown in Figure 2.5.

An upflow roughing filter in layers (URFL) consists of one filter box, with multiple layers of media, ranging from coarse media in the bottom to fine media in the top. The advantage of the URFL is that it has much lower space requirements and capital cost than the HRF or URFS. However, the URFL appears to only be efficient with water sources of low to medium suspended solids concentrations (<150 mg/L) (Galvis et al., 1993). This is due to its smaller solids storage capacity and smaller bed depth than the HRF or URFS. A diagram of the URFL and its design parameters are shown in Figure 2.5.

In general, optimal treatment in roughing filters is achieved by using a higher number of individual compartments (ie. reactors) in series, thus resembling the hydraulic behaviour of a plug flow system (Galvis et al, 1996). Thus, a 3 stage roughing filter is expected to perform better than a 2 stage roughing filter.

Overall, field tests revealed that upflow roughing filters are more efficient in solids removal, as it has a reduced filter depth (in the direction of flow), and smaller space requirements (Wegelin and Schertenleib, 1993). However, since vertical flow (upflow or downflow) roughing filters have a smaller filter depth compared to horizontal roughing filters, it is recommended that vertical flow filters should be limited to pre-treating raw water turbidities less than 150 NTU (Wegelin, 1996).

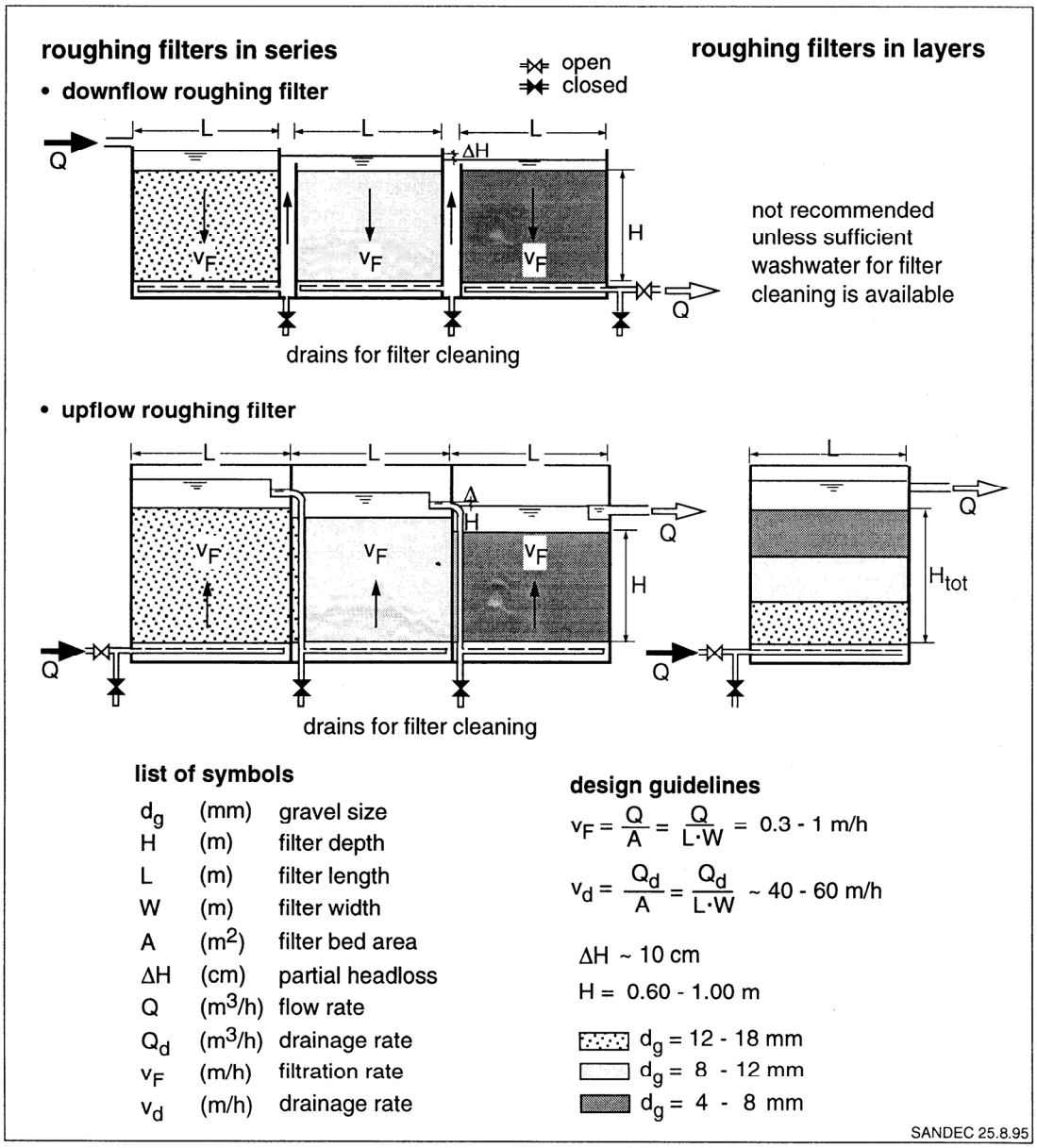

Figure 2.5: Diagram of DRFS, URFS, and URFL and Typical Design Parameters

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2.5.4 Cleaning of Roughing Filters

Roughing filters need to be periodically cleaned to remove accumulated particulate matter and replenish the solids storage capacity of the filter. Roughing filters are cleaned by opening a drain valve at the bottom of the filter and allowing the filter to drain freely under gravity,

thereby flushing solids from the media. The height of roughing filters should be limited to 1.5 m to facilitate easy cleaning (Collins et al., 1994b).

A drainage velocity of 30 m/h (preferably 60-90 m/h) (Wegelin, 1996) is recommended to induce turbulent flow conditions in the media pores, thus dislodging solid deposits from the media. To achieve this drainage velocity, the difference in head between the initial water level in the roughing filter and the drain should be approximately 2.5 m (Galvis et al., 1998). In addition, the drainage pipes should be sized to allow the filter to drain at the recommended velocity. During draining, agitation and dislodgement of the solids can be aided by repeatedly opening and closing the drain valve to send pressure waves through the sand bed.

This method of cleaning has been proven effective, as recovery of initial filter headloss values is very good (Galvis et al., 1996). In fact, Galvis and Visscher (1989) found that cleaning of a roughing filter with a final headloss of 30 cm returned the headloss to 5 to 7 cm. Wegelin et al. (1986) found that filter drainage restores the filter efficiency to near the original value. Furthermore, Rajapakse and Ives (1990) found that draining the roughing filters twice resulted in the removal of greater than 70% of the deposited solids from the filter. Cleaning efficiency, however, can be hindered in filters that treat raw waters that are high in NOM (Collins et al., 1994b). This is most likely due to higher growth of sticky biomass and/or algae, which assist in the attachment and retention of solids on the media grains.

Collins et al. (1994b) examined the suspended solids concentration of the drainage water during filter cleaning. A large initial peak of suspended solids from accumulations in the bottom of the filter was followed by a second smaller peak due to flushing of the solids throughout the depth of the filter. Also, solids were more efficiently flushed from the finer gravel sizes, probably due to higher pore velocities.

The frequency of cleaning is dependent on the loading of particulate matter and biological activity in the filter (Wegelin, 1996), and there are no general recommendations available as water quality conditions are site specific. However, field experience shows that roughing filters are usually cleaned on a weekly to monthly basis. For example, Wegelin (1996)

mentions that cleaning should occur once every one or two weeks during months of high precipitation and runoff, and once every one or two months during dry seasons. Generally, cleaning of roughing filters is not labour intensive. In fact, it is estimated that cleaning can take only 2.4 to 4.6 hours per m^2 of filter area per month (Galvis et al, 1996).

Another method of cleaning is the "upflow method" where an increased upward flow of water induces turbulent conditions in the interstitial pore space and dislodges deposited particles, flushing them from the media. This method is effective in removing solids from the filter, however it is more energy intensive due to pumping requirements. Thus, it may not be suitable for some rural areas where energy supply is limited.

2.5.5 Mechanisms of Removal in Roughing Filtration

The most influential mechanisms of removal in roughing filters are gravitational sedimentation, interception, and diffusion. The principle mechanism, as recognized by many authors, is gravitational sedimentation, where the settling velocity of the waterborne particle is greater than the hydrodynamic forces of the water flow.

Sedimentation and interception are most influential for particles greater than 1 μm, and the removal efficiency due to these mechanisms increases with increasing particle size (Collins et al., 1994b). Diffusion is the most influential removal mechanism for particles less than 1 μm, and the removal efficiency of diffusion increases with decreasing particle size (Collins et al, 1994b).

The roughing filter can be considered a sedimentation basin, where the filter media provides a large surface area and short settling distances for discrete and flocculant particle settling (Wegelin et al., 1986). In conventional sedimentation basins, particles have to reach a settling distance of 1 to 3 metres, whereas in roughing filters, the interstitial settling distance to the gravel surface is only a few millimetres (Wegelin and Schertenleib, 1993). Particles deposit onto media grains in dome-like formations. Eventually, particle accumulations drift and are allowed to migrate freely through the large pore spaces towards the bottom of the roughing filter (Boller, 1993). Thus most particle accumulation occurs in the bottom of the filter.

Compared to rapid filters with smaller media, roughing filters contain less collectors of a larger media size per unit volume, resulting in lower filtration efficiencies (Boller, 1993). Boller (1993) suggests that filter efficiencies similar to rapid filters can only be achieved in media less than 3 mm in diameter, where forces other than gravity become important. However, the advantage of the larger media in roughing filters is that it allows a higher probability of impaction, resulting in higher solids deposits. In addition, particles that are retained in solids deposits will act as additional collectors, increasing the interception of subsequent passing particles (Saidam and Butler, 1996). It is possible that as solids accumulate in the filter and interstitial pore space becomes smaller, interception and straining play an increasingly important role in removal (Saidam and Butler, 1996).

Furthermore, in rapid sand filters, the effects of van der Waals and double-layer forces on particle removal are more significant with decreasing media diameters and increasing particle diameters (Boller, 1993). However, in roughing filters with media diameters generally greater than 5 mm, these forces are negligible (Boller, 1993). Overall, although filtration is not as efficient as rapid filters, solids storage capacity is much larger in roughing filters and headloss develops at a much lower rate, leading to very long filter run times (Boller, 1993).

The secondary mechanisms of particle removal in roughing filters are adsorption to biomass and biological degradation of captured particles (Schulz and Okun, 1984). Organic particles that are retained by the filter are assimilated into a sticky gelatinous biofilm on the media surfaces (Saidam and Butler, 1996). This sticky biofilm, otherwise known as zoogloea, further assimilates organic particles or adsorbs inert particles (Huisman and Wood, 1974). In fact, Collins et al. (1994b) found improved treatment with algae-ripened media compared to clean media. This suggests that the sticky nature of algae assists in particle attachment and solids retention.

It is important to note that, in absence of biomass, adsorption to clean media is not a significant mechanism in roughing filters. Wegelin (1996) found that removals were not greatly influenced by the surface properties of the filter media. This is confirmed by Mbwette

and Wegelin (1989) who found that the shape and surface texture of roughing filter media has a negligible influence on filtration coefficient.

2.5.6 Factors Affecting Removal in Roughing Filters

The principal design parameters for roughing filtration are bed depth, filtration rate, and media size (Collins et al., 1994a). Generally, treatment performance increases with decreasing media size, increasing surface area, decreasing filtration rate, and increasing bed depth. In addition, the characteristics of the particulate matter, such as size and nature (organic or inorganic), have a significant influence on its removal in roughing filters.

Bed depth is the most influential design variable (Collins et al., 1994b). As particles deposit in the filter bed, pore spaces become smaller and the solid deposits are subjected to higher shear forces, causing detachment and penetration of detached solids deeper into the filter bed. Thus, it is important to maximize the bed depth to capture particles that penetrate deeper into the filter.

Filtration rate also has a significant influence on particle removal. Good removals in roughing filters are best achieved with low filtration rates (Boller, 1993). Boller (1993) suggests that low filtration rates are critical to retain particles that are gravitationally deposited or attached to the upper side of the media. It is important to maintain laminar flow conditions in the pores to limit the fluid shear stress on solids deposits. In Figure 2.6, Wegelin (1996) shows that removal efficiency increases with decreasing Reynolds Number (R_e). For example, removal of turbidity was 40% at a R_e of 8, whereas removal was greater than 80% at R_e less than 3.

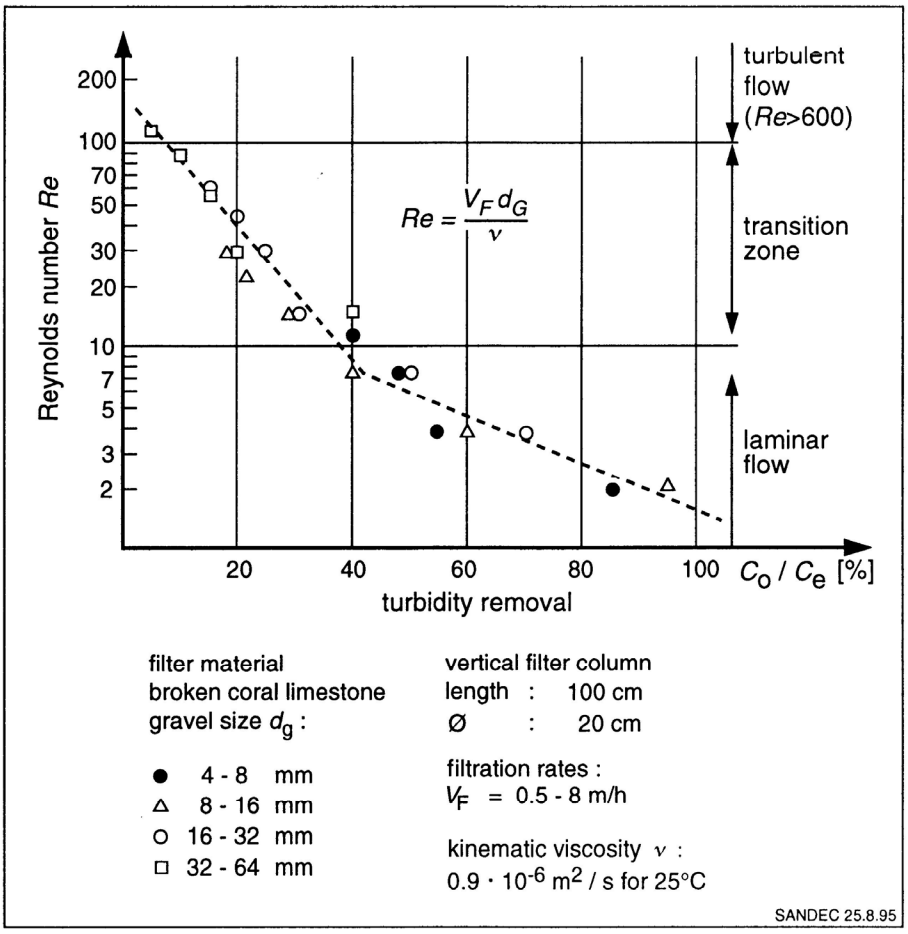

Figure 2.6: Removal Efficiency vs. Reynolds Number in Roughing Filters

(Reprinted by permission of the Swiss Centre for Development Cooperation in Technology and Management (SKAT), from Surface Water Treatment by Roughing Filters: A Design, Construction, and Operation Manual (Sandec Report No. 2/96), by Wegelin, M., SKAT, 1996).

Furthermore, Wegelin et al. (1986) found that at increased filtration rates (2 m/h), coarse particles penetrated deeper into the bed, clogged the finer gravel media, and decreased the filter efficiency, resulting in more particle breakthrough. However, at 0.5 m/h, the bulk of the solid matter was retained by the coarse gravel, leaving the finer gravel sections unloaded. Whereas, at 1 m/h there was a good distribution of solids loading throughout the bed.

When treating high turbidity water, Wegelin (1983) found it necessary to use a filtration rate between 0.5 and 1 m/h to meet the influent requirements for slow sand filtration. In fact, a significant improvement in roughing filter removal efficiency was observed at filtration rates below 2 m/h, compared to over 2 m/h. In addition, Boller (1993) found that the removal of

colloidal particles could only be achieved with filtration rates lower than 2 m/h, preferable lower than 1 m/h.

The media size is another important design variable. Gravity sedimentation on course media is more pronounced than in finer media, where a higher interstitial turbulence limits the gravitational accumulation of particulate matter (Clarke et al., 1996a).

Removal of particulate matter in roughing filters is also dependent on the particle characteristics. Thus, it is important to study the characteristics of the given source water when designing a roughing filter for a particular location. For example, Collins et al. (1994a) found that the removal of mineral particles (more dense, less sticky, and less uniformly sized particles) is most influenced by bed depth, followed by media size and filtration rate, in order of decreasing importance. Conversely, the removal of organic particles similar to algae (less dense, more sticky, more uniformly size particles) is most influenced by filtration rate, followed by media size and filter length in order of decreasing importance.

In addition, Collins et al. (1994a) found that sedimentation of particles was improved in the presence of "sticky" algal particles, which tend to aggregate with other particles and settle out faster. On the other hand, sedimentation of particles was compromised in the presence of stabilizing humic materials.

Furthermore, Wegelin et al. (1986) found that coarse particles are removed more efficiently and only smaller particles penetrate deeper into the roughing filter bed. However, near end of filter run, coarse particles penetrate deeper and smaller particles may breakthrough in the effluent.

The direction of flow can also have an impact on roughing filter performance. Particle trajectory models suggest that horizontal pores are more efficient particle collectors than vertical pores (Boller, 1993). In fact, particles 5 μm in diameter are three to four times more likely to become deposited in horizontal pores than vertical pores. Thus, it is suggested that horizontal filtration is more efficient than vertical filtration in sedimentation. In addition,

Boller (1993) found that the use of smaller media (less than 4 μm in diameter) is not able to reach the same amount of solids deposits as larger media, without incurring a large headloss.

Finally, filtration efficiencies of roughing filters tend to decrease with time as particles are deposited in the filter, resulting in increased interstitial velocity due to decreasing pore space (Boller, 1993). In fact, the removal performance of roughing filters is constant until a critical solids deposit is reached, after which the filter coefficient decreases towards zero and may drop to negative values (Boller, 1993). Negative filter coefficient indicates a breakthrough or "washout" or solids in the effluent. Thus, roughing filters should not be operated past the point of critical solids deposit (Boller, 1993).

Overall, roughing filter performance depends on influent solids concentration, particle-size distribution, media size, bed depth, and filtration rate (Boller, 1993). Boller (1993) suggests that filter design becomes an "art" when attempting to determine the optimal combination of media size and bed depth for a particular source water. The ultimate goal of the design is to distribute the solids loading and headloss development evenly throughout the filter.

2.5.7 Performance of Roughing Filtration

In the following section, performance results from a number of roughing filtration studies are presented. The main parameters discussed are removal of turbidity, suspended solids, colour, DOC, metals, algae, and bacteria. This is followed by a performance comparison of the different types of roughing filters.

2.5.7.1 Removal of Turbidity

Roughing filters are capable of excellent removals of turbidity. Upflow roughing filters can achieve removals between 50 and 90% (Wegelin et al., 1998). The higher removal efficiencies are achievable with higher solids loading (Collins et al., 1994a). The effluent that is produced by roughing filters is well within the limitations of slow sand filtration. For example, Barrett et al. (1991) found that roughing filters reduced the turbidity from 150 NTU down to 15 NTU.

Clarke et al. (1996a) found removals of 60-75% turbidity from a 3-stage URFS, where particles down to a lower measurement limit of 0.75 μm were progressively removed across the filters. It is unclear, however, whether any particles smaller than 0.75 μm were removed. Most impressively, the roughing filter was able to attenuate sudden turbidity peaks and facilitate successful operation of the downstream slow sand filter even when the source water turbidity was continuously greater than 10 NTU for up to 3 months. It was reported that most of the attenuation of the turbidity peaks occurred in the coarsest media fraction (40 mm diameter).

As removal efficiency is a function of the influent loading, significantly low removals can result from treating low turbidity water, especially in the presence of colloidal matter. For example, Galvis et al. (1992) found that for raw water turbidities less than 10 NTU and high amounts of colloidal particles below 0.5 μm, particle removal was only 0-40%. Thus, removal efficiencies in roughing filters are largely dependent on influent loadings.

2.5.7.2 Removal of Suspended Solids

In tropical areas, rivers receiving monsoon runoff can reach suspended solids concentrations of 30,000 mg/L. These solids are mostly inorganic in nature and 80-90% of particles may be below 20 μm in size (Rajapakse and Ives, 1990). Thus, roughing filters are important for protecting slow sand filters from such high solids loadings. Generally, roughing filters can reduce suspended solids concentrations to below 25 mg/L (Rajapakse and Ives, 1990). Wegelin et al (1998) reports removals of 90% with influent concentrations of 50-200 mg/L, and between 50 and 90% with influent concentrations of 5-50 mg/L.

Most impressively, Rajapakse and Ives (1990) reported that, at a filtration rate of 0.72 m/h, roughing filtration reduced an influent suspended solids concentration of 5,000 mg/L to less than 1 mg/L.

2.5.7.3 Removal of Colour

Removal of dissolved colour or true colour in roughing filters has not been well documented. As colour is related to humic substances, it is expected that true colour exists in a relatively stable suspension and is more difficult to remove. Indeed, Collins et al. (1994b) reports that removal of true colour in roughing filters compares favourably to removals achieved by slow sand filtration. Wegelin et al. (1998) reports true colour removals in the range of 20 to 50%.

On the other hand, there have been numerous reports of removals of apparent colour, which is the colour attributable to un-dissolved particulate matter. Wolters et al. (1989) and Barrett et al. (1991) both found removal of apparent colour to be 45 to 80%.

As colour is often used as a surrogate parameter for organic matter, the removal of DOC is expected to compare favourably to removals of true colour. Indeed, Wegelin and Schertenleib (1993) found 15% removal of DOC in roughing filters, which is similar to removals of true colour in roughing filters.

2.5.7.4 Removal of Metals

Removal of metals in roughing filters is not well documented. However, Bernardo (1988) found significant reductions of iron and manganese in roughing filters. Wegelin et al. (1998) reports around 50% removal of iron and manganese. In addition, Wegelin and Schertenleib (1993) found 50% removal of heavy metals in roughing filters.

2.5.7.5 Removal of Algae

Many authors, such as Galvis et al. (1992) and Wegelin and Schertenleib (1993), report operational problems caused by the clogging of slow sand filters with algae. Thus, roughing filters can play an important role in reducing the algal load on slow sand filters and increasing the filter run length. Barrett et al. (1991) found that algal removal in roughing filters is in the range of 30-80%.

2.5.7.6 Removal of Bacteria

Traditionally, roughing filters have not been viewed as valuable contributors to the removal of bacteria. This is because their original purpose was to reduce solids loading on slow sand filters. However, in recent decades the microbiological performance of roughing filters has been recognized.

The results show that roughing filters are not only important from an operational point of view, but also integral in providing a robust approach to water treatment. For example, Clarke et al. (1996b) found that fecal coliform removals were in the range of 80-90%, and suggested that roughing filters play a significant role in the multi-barrier approach to pathogen removal.

Wegelin et al. (1998) reports removals of fecal coliforms in the range of 0.65 to 2.5 log units. The higher removals are achieved with higher levels of bacterial contamination (Wegelin et al., 1998). For example, Barrett el al. (1991) found peak coliform bacteria removals of 90%, and found that higher reductions were associated with higher influent turbidity loadings. Wegelin and Schertenleib (1993) found even higher bacteria removals of 90 to 99%.

In addition, performance does not seem to be affected by sudden changes in influent conditions. For example, Clarke et al. (1996a) found that removal of bacteriophage was similar under sudden surges of influent turbidity, compared to during periods of stable influent turbidity.

Generally, roughing filters are capable of reducing influent bacteria to levels that are easily treatable with slow sand filtration. For example, Barrett et al. (1991) studied an upflow roughing filter that reduced coliforms from 16,000 colonies/100 mL down to 1,680/100 mL. Furthermore, Wegelin and Schertenleib (1993) studied a HRF that reduced *E. Coli* from 200-1,000/100 mL to 10-30/100 mL.

Overall, roughing filters are important for adding robustness to the process of bacteria removal in multistage filters. Not only do they protect the slow sand filter from premature

clogging and potential pathogen breakthrough, they provide significant pre-treatment of bacteria, and reduce influent contamination to levels that are easily treatable by slow sand filtration.

2.5.7.7 Comparison of Different Roughing Filter Types

Galvis et al. (1996) compared upflow and horizontal flow roughing filters using water from a heavily polluted lowland river in Columbia. Raw water turbidities ranged from 15 to 1,880 NTU, true colour ranged between 24 and 344 TCU, and fecal coliform counts ranged between 7,300 and 396,000 FCU/100 mL. Each roughing filter had a total bed length of 4.3 m and a filtration rate of 0.7 m/h. The HRF achieved turbidity, suspended solids, and fecal coliform removals of 66.7%, 93.8%, and 95.6%, respectively. In comparison, the UFRS achieved turbidity, suspended solids, and fecal coliform removals of 80%, 97.9%, and 99.4%, respectively.

For the given water source, the upflow roughing filter out-performed the horizontal roughing filter. In this case, the URFS may be preferred over the HRF due to its lower cost and space requirements. However, the URFS may need to be cleaned more frequently than the HRF, but because the cleaning of roughing filters is not labour or energy intensive, a URFS could be the optimal choice for the given site.

In another study, Galvis et al. (1993) compared the performance of a HRF, URFS, and an URFL with equal hydraulic retention times. The removal of fecal coliforms in each filter was 93.4%, 99.5%, and 95.1%, respectively. In each case, the final effluent after a subsequent stage of slow sand filtration resulted in 4.9-5.5 log removal of fecal coliforms. With these effluent qualities, any use of post-disinfection is essentially a redundant safety barrier.

2.5.8 Performance of Multistage Filtration

2.5.8.1 The Success of Multistage Filtration in Columbia

Multistage filtration (MSF) has had particular success in Columbia where there are about 50 full scale systems in operation, and 10 of them have been in operation since the 1980s (Galvis

and Visscher, 1999). In most of these systems, the local consumers of the water supply cover the low operational and maintenance costs.

Galvis and Visscher (1999) compared the performance of two different MSF systems in Columbia, one of them treating water from a heavily polluted river and the other treating a less polluted surface water. Both MSF systems provided similar effluent levels of turbidity, fecal coliforms, and colour. This suggests that MSF treatment "can adapt itself to the type of raw water and the concentration of contamination" (Galvis and Visscher, 1999), and higher removal efficiency results when using raw water sources that are more polluted. For example, MSF achieved a 2.5 log removal of fecal coliforms with raw water levels of 330 FCU/100 mL. In contrast, MSF achieved a 4.6 log removal of fecal coliforms with raw water levels of 44,500 FCU/100 mL (Galvis and Visscher, 1999).

One of the main limitations of slow sand filtration is its inability to effectively treat high levels of colloidal matter and true colour. Galvis and Visscher (1999) studied the efficiency of MSF in removing colour from heavily polluted surface water, with raw water colour ranging between 63 and 93 TCU. Due to the high raw water colour, removal efficiencies were high (between 86 and 89%), resulting in a SSF effluent between 8.2 and 12 TCU, and below 15 TCU between 77 and 96% of the time.

In addition, Galvis and Visscher (1999) report filter run times of 46 to 178 days, which far exceed slow sand filter run times of one to four weeks.

2.5.8.2 The Success of Multistage Filtration in North America

Recently, there has been increasing interest and use of MSF for surface water treatment in small communities throughout North America. LeCraw et al. (2004) documents the performance of MSF in a number of onsite pilot studies and full-scale plants. The process includes pre-ozonation for colour and organics removal, roughing filtration, and slow sand filtration. In some cases, a post-GAC filter was used for additional removal of colour and organics. Generally, the process has been used with raw water turbidities and colour ranging from <1 to 10 NTU (with spikes greater than 30 NTU) and <5 to 60 TCU, respectively

(LeCraw et al., 2004). Typical effluent turbidities and colour ranged from <0.1 to 0.2 NTU and <5 TCU, respectively.

A summary of the results from several MSF systems, both pilot scale and full scale, is shown in Table 2.4. In each location, the effluent turbidity and colour was lower than the regulatory requirements of the SWTR. These results are promising for small communities that are looking for a simple and cost effective treatment method. Thus far, MSF has proven to be a reliable treatment technology in several cold climate locations throughout North America.

Table 2.4: Summary of MSF Performance from LeCraw et al. (2004)

Location	Pilot or Full-Scale?	Applied O_3 Dose (mg/L)	RF Filtration Rate (m/h)	SSF Filtration Rate (m/h)	Raw Water Quality		Effluent Water Quality		
					Turbidity (NTU)	Colour (TCU)	Turbidity (NTU)	Colour (TCU)	TOC Removal
Sturgeon Lake, Ontario	Pilot			0.3	1.5 to 10		<0.5		
North Haven, Maine	Pilot	3	1.8	0.3	1 to 5	20	0.2 to 0.4	<5	
White River, Ontario	Pilot	2 to 4	1.8	0.3 to 0.6	20 to 40	25	0.5 to 2	5	30 to 35%
North Haven, Maine	Full-Scale	4.3	1.8	0.29	1 to 5	10 to 40	<0.2	<5	24 to 34%
Wabauskang First Nation	Full-Scale	2.5	1.8	0.3	1 to 3	10 to 20	<0.3	<5	

RF = roughing filter; SSF = slow sand filter

2.6 CONCLUSIONS

Throughout the world and North America, small communities in rural areas often rely on contaminated surface water for their drinking water supply. As a result, numerous water-related outbreaks due to inappropriate small system treatment technologies have led to a growing need for a simple, reliable, and sustainable treatment technology.

Slow sand filtration is a proven, economical, and sustainable technology for rural drinking water treatment. It is a simple technology that is capable of achieving high standards of treatment without the use of chemicals and labour intensive operations and maintenance. However, the process is highly sensitive to raw water turbidity where high turbidity levels can lead to frequent clogging, decreased filter run length, and potential pathogen breakthrough. Furthermore, the process is poor at removing colloidal matter, colour, organics, THM precursor material, and is less effective at lower temperatures.

Multistage filtration is a proven method to cope with these treatment limitations, and is compatible with small community needs. It is a robust, multi-barrier treatment method, which traditionally consists of pre-treatment with roughing filtration followed by slow sand filtration. Roughing filtration reduces the solids loading on slow sand filters, which facilitates longer filter runs and optimal slow sand filter performance. The use of additional pre-ozonation and post-granular activated carbon filtration stages is effective in removing colour, organics, and THMFP. Overall, particulate matter and pathogenic microorganisms face a series of treatment barriers throughout the treatment system. Thus, multistage filtration can consistently provide an effluent water quality that exceeds the capabilities and limitations of slow sand filtration alone.

Multistage filtration has been particularly successful in tropical areas, such as Columbia, treating extremely contaminated surface water with high solids loadings. It has also been successful in a limited number of installations in North America. However, it has yet to be challenged in treating highly variable water qualities at both decreased water temperatures and increased hydraulic loading rates. By investigating its performance under a range of raw water qualities, temperatures, and hydraulics loadings, a range of small system conditions could be established, for which multistage filtration is a viable drinking water treatment option.

This type of research would further elucidate the potential of multistage filtration technology as a reliable and robust drinking water treatment method for small communities throughout the world, especially in northern climates.

Chapter 3

3 Materials and Methods

3.1 EXPERIMENTAL APPARATUS

This study involved the testing of two pilot-scale multistage filtration systems using raw surface water from the Grand River in Southern Ontario, Canada. For a background on the Grand River and its water quality, refer to Chapter 4. Both pilot systems were sheltered from direct sunlight inside the Region of Waterloo's Mannheim Water Treatment Plant raw water intake building, otherwise known as the Hidden Valley Low Lift Pumping Station, in Kitchener, Ontario. The raw water intake building is located on the north bank of the Grand River, just upstream of a spillway dam. The location of the intake is shown in Figure 3.1.

Figure 3.1: Raw Water Intake at Grand River

Raw water from the Grand River entered the building through an intake pipe located just upstream from the dam. Following pre-screening to remove course debris, a small flow of water was diverted from the main flow, and pumped to supply the both pilot systems.

Both pilot multistage filter designs included an upflow roughing filter followed by slow sand filtration. However, each pilot system was unique with respect to its filter design parameters, a summary of which is given in Table 3.1. Furthermore, both pilot systems featured additional stages of treatment, which are explained in detail in the following section.

Table 3.1: Summary of Pilot Multistage Filter Design Parameters

Design parameter	**Recommended by Literature**	**Pilot system 1**	**Pilot system 2†**	
			A	B
Roughing Filter:				
Diameter of media layer 1 (mm)	12 - 18 *	8 - 12.5	12.7 - 19.1	9.5 - 12.7
Diameter of media layer 2 (mm)	8 - 12 *	2.36 - 3.35	9.5 - 12.7	4.8 - 9.5
Diameter of media layer 3 (mm)	4 - 8 *	0.85 - 1.2 (GAC)	4.8 - 9.5	1.5 - 3.2
Depth of media layer 1 (m)	0.2 - 0.3 **	0.1	0.5	0.1
Depth of media layer 2 (m)	0.15 - 0.2 **	0.2	0.4	0.55
Depth of media layer 3 (m)	0.15 - 0.2 **	0.3	0.3	0.55
Total bed depth (m)	0.9 - 1.2 **	0.6	1.2	1.2
Diameter of filter column (m)	n/a	0.16	0.2	0.2
Area of filter column (m^2)	n/a	0.020	0.031	0.031
Filtration rate (m/hr)	0.3 - 1.5 *	0.7 - 3.0	0.47 - 1.35	0.47 - 1.35
Empty bed contact time (hr)	n/a	0.2 - 0.86	0.89 - 2.55	0.89 - 2.55
Slow Sand Filter:				
Effective diameter of media (mm)	0.15 - 0.35 ***	0.35	0.37	0.37
Uniformity coefficient of media	< 2.0 ***	1.7	1.67	1.67
Bed depth (m)	0.7 - 1.2 ***	0.45	1	0.5
Diameter of filter column (m)	n/a	0.315	0.295	0.15
Area of filter column (m^2)	n/a	0.078	0.068	0.018
Filtration rate (m/hr)	0.1 - 0.4 ***	0.2 - 0.8	0.2 - 0.6	0.2 - 0.6
Empty bed contact time (hr)	n/a	0.56 - 2.25	1.67 - 5.0	0.83 - 2.5

† A = roughing filter A or slow sand filter 1
B = roughing filter B or slow sand filter 2

* Wegelin, 1996
** Galvis et. al., 1998
*** Huisman and Wood, 1974

3.1.1 Description of Pilot System 1

The first pilot system, referred to as pilot system 1, is a contemporary multistage filter design that has evolved from numerous modifications resulting from previous field trials conducted by MS Filter Inc. The system was contained in a portable trailer, which was constructed by MS Filter Inc. and consists of two identical and independent treatment trains, named train 1 and train 2. A photo of the interior and exterior of the trailer is shown in Figure 3.2 and Figure 3.3, respectively.

Figure 3.2: Pilot System 1 - Interior View

Figure 3.3: Pilot System 1 – Exterior View

As shown in Table 3.1, pilot system 1 featured shallower bed depths in the roughing filter (0.6 m) and slow sand filter (0.45 m) than recommended in the literature (~1 m), due in part to the size constraints of the trailer in which it was located and economical optimization of the process. The range of slow sand filtration rates tested in pilot 1 (0.2 to 0.8 m/h) was larger than recommended in the literature (0.1 to 0.4 m/h) for economical reasons (ie. reducing the footprint of the plant) and to simulate the scenario of doubling the flow rate through one of two parallel systems while the other system is taken offline for cleaning. The media sizes used in the three layers of gravel in the roughing filter were also smaller than recommended for increased filtration efficiency. In addition, the roughing filter featured a novel modification, the use of granular activated carbon (GAC) for the fine media layer. Besides reducing the turbidity entering the slow sand filter, the layer of GAC would also function to adsorb any residual ozone in the water before entering the biologically active slow sand filter.

Furthermore, pilot system 1 featured an additional stage of pre-ozonation prior to the roughing filter, and a GAC filter following the slow sand filter. The main purpose of pre-ozonation was for the removal of dissolved organic carbon, colour, and disinfection by-

product formation potential, and to render natural organic matter to a more readily biodegradable form for subsequent biological removal in the slow sand filter. The main purpose of the GAC filter was for additional removal of dissolved organic carbon, colour, and disinfection by-product formation potential. A detailed process diagram of pilot system 1 is shown in Figure 3.4.

Raw water entered pilot system 1 at a pressure of 50 psi and a flow of 8 Lpm was diverted to each train. The water then entered the bottom of the ozone contactor after passing through an injector where ozone gas was applied at a dose of approximately 8 mg/L. Ozone gas was generated on-site with an Azcozon SNOA-4 ozone generator. The water travelled up through the ozone contactor (shown in Figure 3.5), which was filled with PVC saddles functioning as contact media to facilitate the contact of ozone gas with the water. Assuming a transfer efficiency of 50%, the estimated aqueous ozone dose was 4 mg/L.

At the top of the ozone contactor, a portion of the flow was collected and pumped to feed the rest of the MSF system using a Masterflex® peristaltic pump, shown in Figure 3.6. Flow was controlled by adjusting the speed of the pump, and measured using Fabco variable area rotameters with polysulfone body and PVC and stainless steel floats, shown in Figure 3.7.

After the pumping stage, water entered the top of the secondary ozone contactor, which acted as a plug flow reactor to provide extra time for the ozone to react with the water before entering the bottom of the roughing filter.

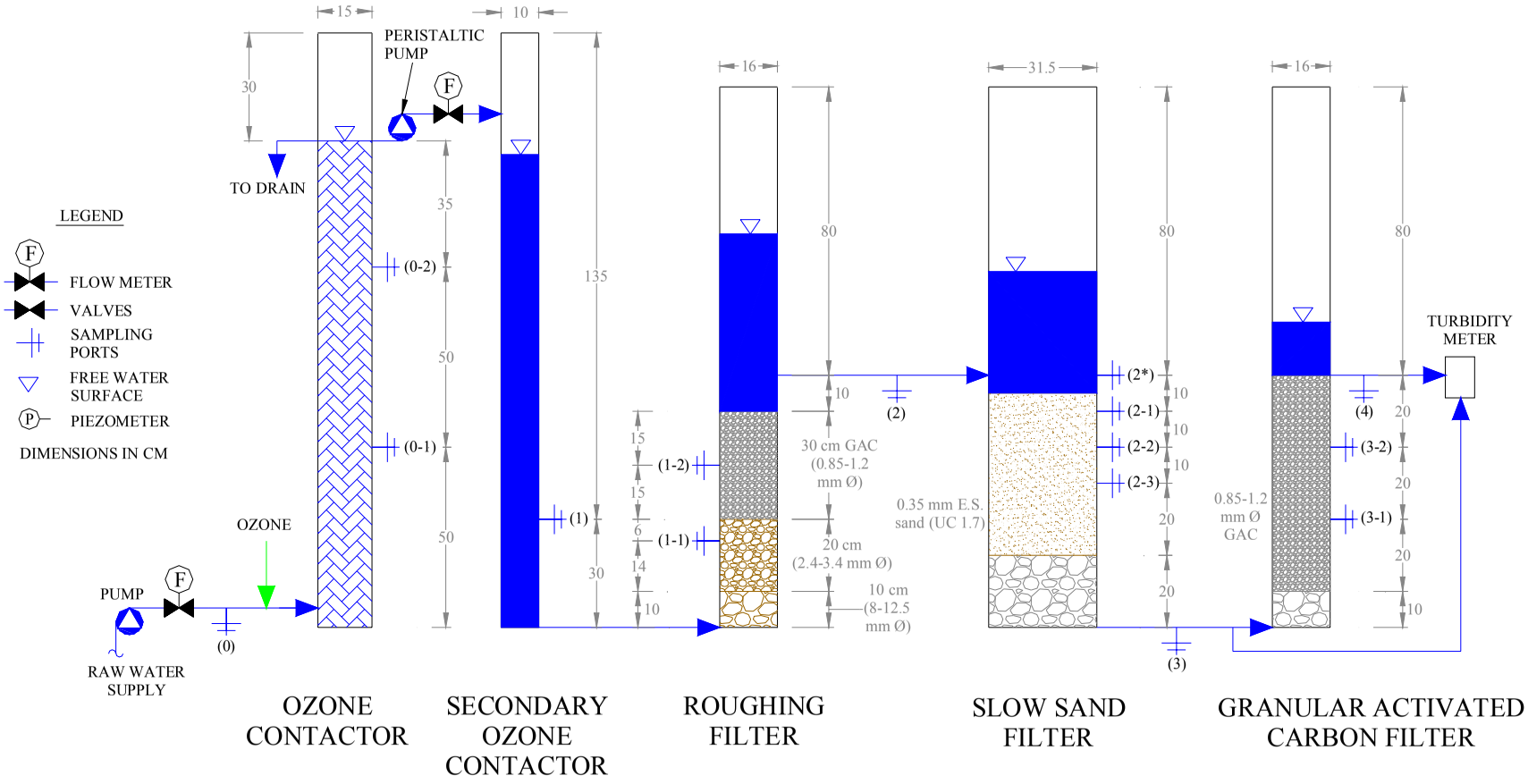

Figure 3.4: Process Diagram of Pilot System 1

Figure 3.5: Ozone Contactor

Figure 3.6: Peristaltic Pumps

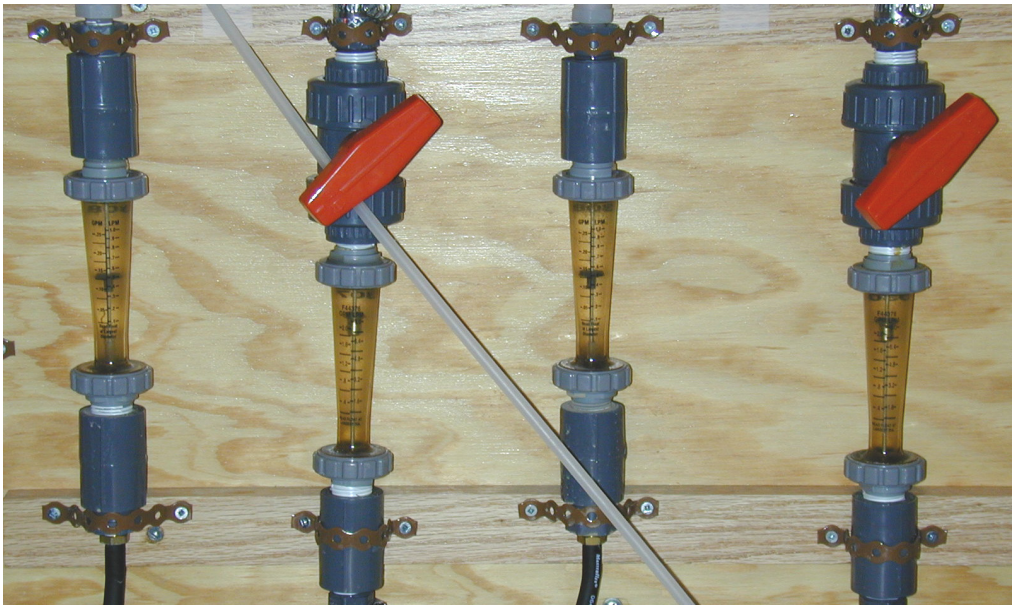

Figure 3.7: Flow Meters – Pilot System 1

In the roughing filter, water flowed upward through a series of media layers: 10 cm depth of 8-12.5 mm diameter gravel, 20 cm of 2.4-3.4 mm gravel, and 30 cm of 0.85-1.2 mm GAC. Besides reducing the turbidity entering the slow sand filter, the layer of GAC could also function to adsorb any residual ozone in the water before entering the biologically active slow sand filter. However, due to the ozone demand of solids accumulations in the bottom of the roughing filter, any residual ozone entering the roughing filter would quickly be depleted, thus allowing biological activity to flourish throughout the rest of the filter, which is desirable from a treatment point of view.

After the roughing filter, water entered the slow sand filter and flowed downward through a 45 cm depth of 0.35 mm effective diameter sand media. The sand was supported by three layers of underdrain gravel: 5 cm of 0.85-1.2 mm diameter gravel, 5 cm of 2.4-3.4 mm gravel, and 10 cm of 8-12.5 mm diameter gravel.

The outlet pipe of the sand filter rose to an elevation of 5 cm above the sand bed to prevent negative pressures in the sand bed, which result in the formation of air bubbles (Barrett et al, 1991). At this elevation, the water was open to the atmosphere with air relief piping to

prevent siphoning of the filter bed. This also prevented the water level from dropping below the top of the sand bed and desiccating the biological layer in the event of a shutdown.

After the slow sand filter, water entered the bottom of the GAC filter. Water flowed upward through a 60 cm depth of GAC, which was supported by 10 cm of 2.4-3.4 mm underdrain gravel. After leaving the top of the GAC filter, the effluent was returned to the river.

The filter columns were constructed of PVC piping and insulated with foam insulating wrap, which also sheltered the filters from artificial indoor lighting. The diameter of the slow sand filter was designed using a d_{column}/d_{media} ratio of approximately 800, and the roughing filter was designed using an arbitrarily chosen minimum d_{column}/d_{media} ratio of 10. Wegelin (1996) reported that a d_{column}/d_{media} ratio of less than 25 was recommended for roughing filters, as sidewall short-circuiting is generally less of a concern in course gravel.

The piping and valve materials consisted of rigid PVC, Teflon tubing, and laboratory grade clear PVC tubing. Viton® tubing was used in the peristaltic pumping stage. Water sampling ports were made of PVC, as shown in Figure 3.8.

Figure 3.8: Typical Sampling Ports in Pilot System 1

3.1.2 Description of Pilot System 2

The second pilot system, referred to as pilot system 2, was designed and constructed by the author. The main stages of the process, the upflow roughing filter and slow sand filter, were designed according to recommendations in the literature, and were approximately twice the bed depth of that in pilot system 1. The range of slow sand filtration rates (0.2 to 0.6 m/h) was also lower and more practical than that of pilot system 1 (0.2 to 0.8 m/h). Pilot 2 also featured an additional roughing filter of modified design for increased filtration efficiency and a second slow sand filter in series for increased robustness and protection against pathogen breakthrough. A photo of the pilot system 2 is shown in Figure 3.9.

Figure 3.9: Pilot System 2

The additional roughing filter (roughing filter B) operated in parallel with the original roughing filter (roughing filter A), to provide a comparison between the two filter designs. Roughing filter B used finer media sizes and larger bed depths to provide increased filtration efficiency in terms of the L/d ratio. An explanation of L/d ratio and its impact on the design of roughing filter B is given in Appendix A.

The concept of the additional slow sand filter was a contribution by Daniel Urfer, who tested a similar system and installed a full-scale system in Switzerland. Its main purpose was to provide increased robustness and protection against potential pathogen breakthrough during high turbidity events, cleaning of the first slow sand filter, increased hydraulic loadings, and colder temperatures. A detailed process diagram of pilot system 2 is shown in Figure 3.10.

In pilot system 2, raw water first entered the constant head tank, shown in Figure 3.11, where a constant water level was maintained with a V-notch weir. The purpose of the constant head tank was to ensure the system was fed water with a constant water pressure, thus eliminating the need for a pump. The difference in elevation between the water level in the constant head tank and the water level in the roughing filter was 1.3 m. After the constant head tank, the flow was divided to each roughing filter. The flow to each roughing filter was controlled with a needle valve and measured with a Kobold KSK plastic flow meter with PVDF body and float.

Water entered the bottom of each roughing filter and flowed upward through a series of media layers. Roughing filter A, designed to recommendations in the literature, consisted of a 50 cm depth of 12.7-19.1 mm diameter gravel, 40 cm of 9.5-12.7 mm gravel, and 30 cm of 4.8-9.5 mm gravel. Roughing filter B, a modified roughing filter design, consisted of 10 cm of 9.5-12.7 mm gravel, 55 cm of 4.8-9.5 mm gravel, and 55 cm of 1.5-3.2 mm gravel. The effluent from either roughing filter could be used to feed the rest of the process, while effluent from the other filter would be diverted to waste.

After the roughing filter, water entered the top of slow sand filter 1 and flowed downward through a 100 cm depth of sand media with an effective diameter of 0.37 mm. The sand was supported by three layers of underdrain gravel: 5 cm of 1.5-3.2 mm diameter gravel, 5 cm of 4.8-9.5 mm gravel, and 5 cm of 12.7-19.1 mm gravel.

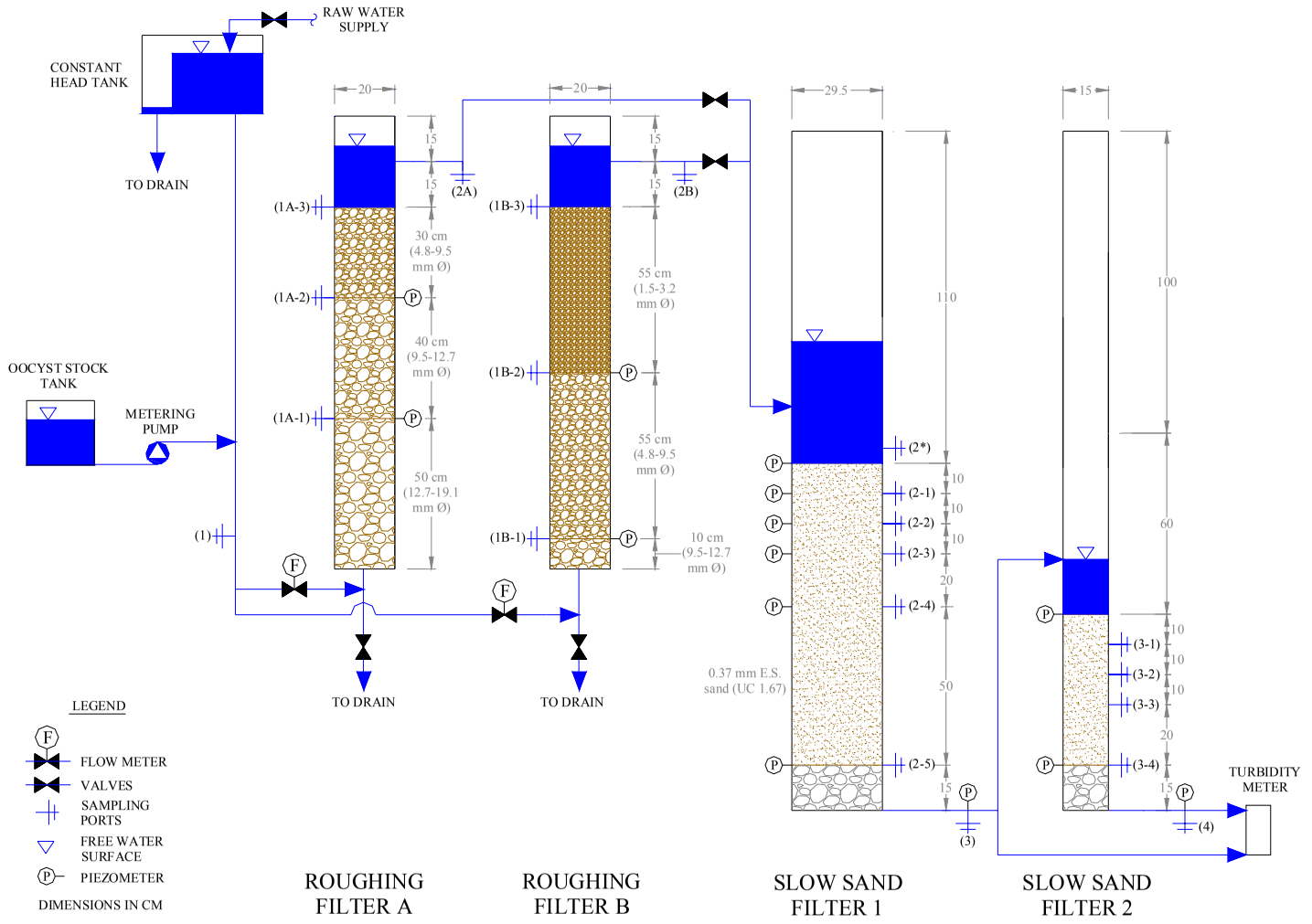

Figure 3.10: Process Diagram of Pilot System 2

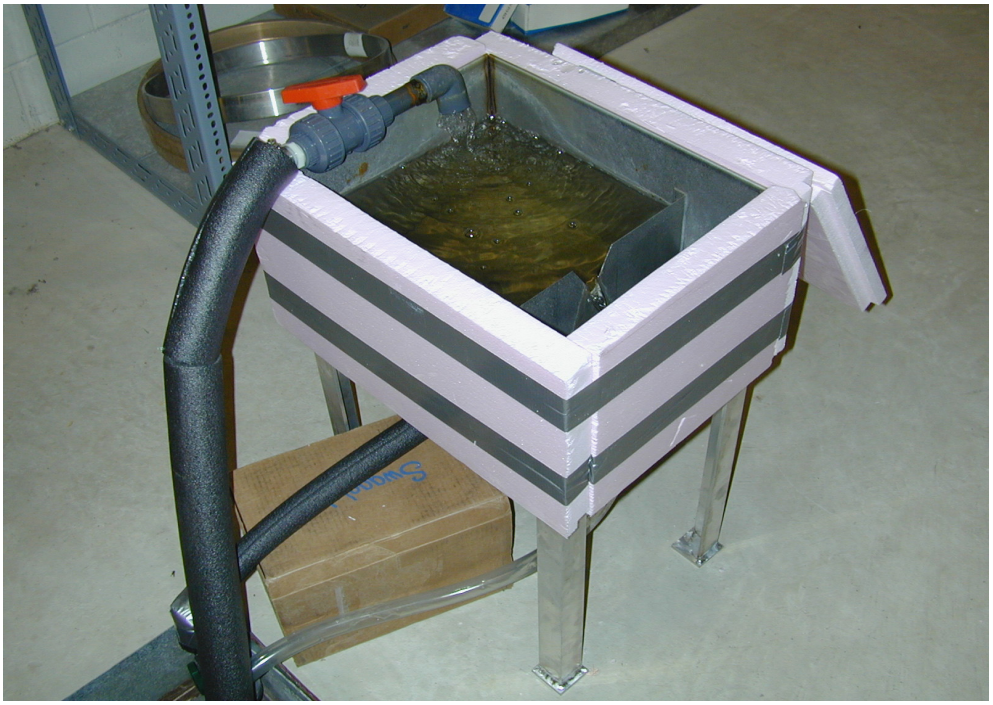

Figure 3.11: Constant Head Tank

After slow sand filter 1, water entered the top of slow sand filter 2. For economical reasons, slow sand filter 2 was downsized to half the diameter of slow sand filter 1. Thus, to achieve the same hydraulic loading rate in both slow sand filters, it was necessary to divert a portion of the flow to waste before entering slow sand filter 2.

After entering slow sand filter 2, water flowed downward through a 50 cm depth of sand media with an effective diameter of 0.37 mm. Three layers of underdrain gravel, similar to slow sand filter 1, supported the sand. After leaving the bottom of slow sand filter 2, the effluent was returned to the river.

The outlet pipe of each sand filter rose to an elevation of 5 cm above the sand bed to prevent negative pressures in the sand bed, which result in the formation of air bubbles (Barrett et al, 1991). At this elevation, the water was open to the atmosphere with air relief piping to prevent siphoning of the filter bed. This also prevented the water level from dropping below the top of the sand bed and desiccating the biological layer in the event of a shutdown.

The filter columns were constructed of PVC piping. The diameter of the slow sand filter was designed using a d_{column}/d_{media} ratio of approximately 800, and the roughing filter was designed using an arbitrarily chosen minimum d_{column}/d_{media} ratio of approximately 10. Wegelin (1996) reported that a d_{column}/d_{media} ratio of less than 25 was recommended for roughing filters, as sidewall short-circuiting is generally less of a concern in course gravel.

The piping consisted of mostly Teflon tubing connected with stainless steel Swagelok® fittings and valves. Both the filter columns and piping were insulated with polyfoam insulation, which also sheltered the filters from artificial indoor lighting. Water sampling ports consisted of laboratory grade Tygon® tubing, nylon or polypropylene fittings, and tubing clamps, as shown in Figure 3.12. Water levels were measured with piezometers, which consisted of standard clear PVC tubing, as shown in Figure 3.13.

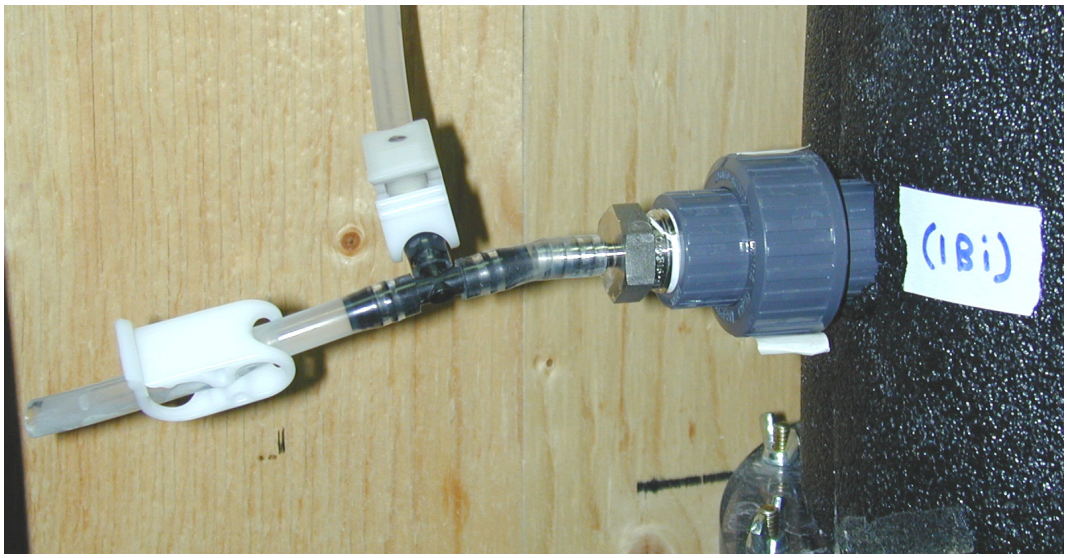

Figure 3.12: Typical Sampling Port – Pilot System 2

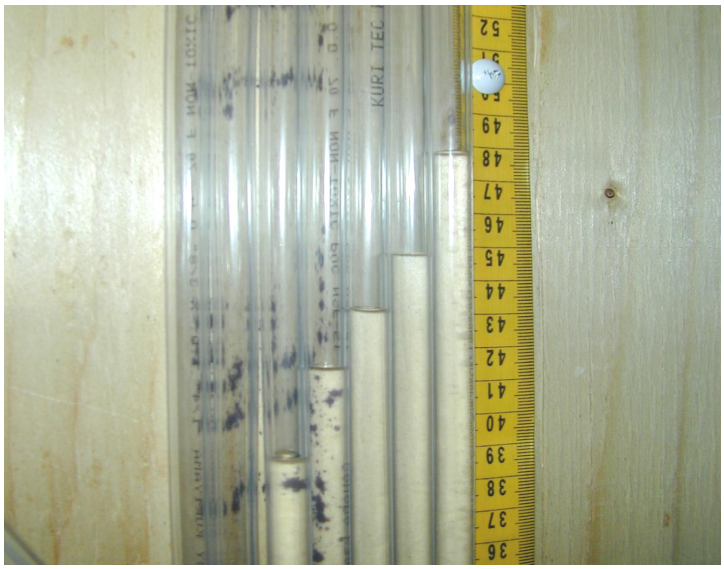

Figure 3.13: Piezometers

3.1.3 Media Specifications

The filter media used in both pilot systems was in accordance with the ANSI/AWWA B100-01 Standard for Filtering Material. The slow sand filters in pilot system 1 and 2 used a similar effective media diameter of 0.35 mm and 0.37 mm, respectively. This is the maximum recommended size of media for slow sand filtration, and it was chosen for this study to maximize filter run times and allow increased pore size for biomass accumulation in the upper portion of the sand bed. However, this could cause potential problems with cleaning and removal of headloss from the sand bed, as a larger pore size would allow deeper penetration of schmutzdecke into the bed. To counteract these potential effects, the uniformity coefficient (UC) of the sand in both systems was kept to a minimum (approximately 1.7), resulting is a relatively smaller range of media sizes. The sand media in both systems had a specific gravity of approximately 2.6 and acid solubility of less than 5%.

Due to the similarity in media size and uniformity coefficient between the two pilot systems, it was assumed that similar slow sand filter media was used in both pilot systems, even though they were obtained from two different sources. Thus, any differences in slow sand filter performance between the two systems were not attributed to the media characteristics.

3.2 EXPERIMENTAL DESIGN

The objective of the experimental design was to test the performance of each pilot multistage filter under challenging operating conditions and determine the impact of varying experimental factors on treatment performance. The major experimental factors that were investigated in this study include:

- Hydraulic loading rate (through the slow sand filter)
  - Pilot system 1 – 0.2 m/h, 0.4 m/h, and 0.8 m/h (train 2 only)
  - Pilot system 2 – 0.2 m/h, 0.4 m/h, and 0.6 m/h
- Influent water temperature – 2 to 20^{o}C (ambient)
- Influent water turbidity – 1 to >100 NTU (ambient)
- *Cryptosporidium* challenge tests – influent spikes of 10^{6} oocysts/L
- Media size (d) and bed depth (L) of roughing filters
- The presence/absence of pre-ozonation for removal of coliform bacteria
- The use of post-granular activated carbon filtration on removal of turbidity
- The impact of a second slow sand filter in series for increased robustness and protection against pathogen breakthrough

In addition, this investigation also analyzed treatment performance during a number of dynamic events that occurred throughout the duration the scheduled tests. They were:

- Filter acclimatization (at startup)
- Stable operation (periods of stable water quality and hydraulic loading)
- Spring runoff and/or rain events of high turbidity
- Operational disturbances (temporary surge in hydraulic loading, cleaning, etc.)

There were a total of 11 scheduled tests in this study, each with a defined objective. The testing schedule and objective of each test is shown in Table 3.2. The operating conditions for each test are outlined in Table 3.3.

During each test, treatment performance was analyzed by collecting water samples from various sampling ports throughout each pilot system. The locations of the sampling ports for pilot system 1 and 2 are shown in Figure 3.4 and Figure 3.10, respectively. Water samples were analyzed either onsite or at University of Waterloo laboratory, or at a commercial lab. A list of the water quality parameters that were measured in this study, including the frequency and sampling location of each parameter, is given in Table 3.4.

Table 3.2: Testing Schedule

Test #	Objective	Dates
1	Determine the performance of pilot system 1 during higher than normal hydraulic loading rate (0.4 m/h*)	Feb9-Mar6, Mar15-Mar29, Apr9-Jun1
2	Determine the performance of pilot system 2 during higher than normal hydraulic loading rate (0.4 m/h*)	Feb21-Mar9, Mar15-Mar19, Apr11-Jun1
3	Determine the performance of pilot system 1 (train 2) under high hydraulic loading rate (0.6 to 0.8 m/h*)	Apr14-Apr21 (0.6 m/h), Apr21-Apr30 (0.8 m/h), Apr30-May3 (0.6 m/h), May3-May25 (0.8 m/h)
4	Determine of the performance of pilot system 2 under high hydraulic loading rate (0.6 m/h*)	Mar19-Apr11
5**	Determine the performance of pilot system 1 and 2 in removing *C. parvum* (*Cryptosporidium* challenge tests)	Apr28-Jun7
6	Determine the performance of pilot system 1 during runoff events of high raw water turbidity (0.2 m/h*)	Mar6-Mar15, Mar 29-Apr9
7	Determine the performance of pilot system 2 during runoff events of high raw water turbidity (0.2 m/h*)	Mar9-Mar15
8	Determine the effect of roughing filter design on the performance of pilot system 2	Feb9-May10
9	Determine the effect of pre-ozonation in pilot system 1	Feb25-Mar22
10	Determine the effect of granular activated carbon filtration in pilot system 1	Feb25-Apr26 (not continuous)
11	Determine the effect of an additional slow sand filter in pilot system 2	Feb9-May10

* hydraulic loading rate through the slow sand filter

** for a detailed breakdown of the *Cryptosporidium* challenge tests, see Table 3.6

Table 3.3: Filter Operating Conditions

Test	Pilot	Flow (Lpm)	RF filtration rate (m/h)	SSF filtration rate (m/h)	GAC filter filtration rate (m/h)	Pre-ozonation	Location of online turbidimeter
1	1 (T1)	0.5	1.5	0.4	1.5	yes	after SSF and GAC filter
	1 (T2)	0.5	1.5	0.4	1.5	yes	after SSF and GAC filter
2	2	0.48	0.9	0.4	n/a	no	after SSF1 and SSF2
3	1 (T1)	0.5	1.5	0.4	1.5	yes	after SSF and GAC filter
	1 (T2)	1	3	0.8	3	yes	after SSF and GAC filter
4	2	0.72	1.35	0.6	n/a	no	after SSF1 and SSF2
5a	1 (T1)	0.5	1.5	0.4	n/a	yes	after SSF
5b	1 (T2)	1	3	0.8	n/a	yes	after SSF
5c	2	0.48	0.9	0.4	n/a	no	after SSF1
6	1 (T1 & T2)	0.25	0.7	0.2	n/a	yes	after SSF and GAC filter
7	2	0.25	0.47	0.2	n/a	no	after SSF1 and SSF2
8	2	0.25 to 0.72	0.47 to 1.35	n/a	n/a	no	n/a
9	1 (T1)	0.25 to 0.5	0.7 to 1.5	0.2 to 0.4	0.7 to 1.5	yes	after SSF and GAC filter
	1 (T2)	0.25 to 0.5	0.7 to 1.5	0.2 to 0.4	0.7 to 1.5	no	after SSF and GAC filter
10	1 (T1 & T2)	0.25 to 1	0.7 to 3	0.2 to 0.8	0.7 to 3	yes	after GAC filter
11	2	0.25 to 0.72	0.47 to 1.35	0.2 to 0.6	n/a	no	after SSF2

Notes:
T1 - Train 1
T2 - Train 2
RF - Roughing Filter
SSF - Slow Sand Filter
SSF1 - Slow Sand Filter 1 (Pilot 2 only)
SSF2 - Slow Sand Filter 2 (Pilot 2 only)

Table 3.4: Summary of Water Quality Parameters

Water Sampling Parameters	Units	Frequency	Sampling Location (Pilot 1)	Sampling Location (Pilot 2)
Turbidity (NTU)	NTU	twice/week & online	0, 1, 1-1, 1-2, 2, 2-1, 2-2, 2-3, 3, 3-1, 3-2, 4	1, 1A-1, 1A-2, 1B-1, 1B-2, 2A, 2B, 2-1, 2-2, 2-3, 2-4, 3, 3-1, 3-2, 3-3, 4
Water head	m	3 times/week	1, 2	1, 1A-1, 1A-2, 1B-1, 1B-2, 2, 2-1, 2-2, 2-3, 2-4, 3, 4
Temperature	^{o}C	twice/week	0, 1, 2, 3, 4	1, 2A, 2B, 3, 4
Total coliforms	#/100mL	weekly	0, 3 (1, 2 - biweekly)	1, 3 (2A, 2B, 4 - biweekly)
Fecal coliforms	#/100mL	weekly	0, 3	1, 3
Cryptosporidium parvum	oocysts/100mL	3 challenge tests	2*, 3	2*, 3, 4

Legend:
0 - ozone contactor influent (pilot 1)
1 - roughing filter influent
1-1, 1-2 - roughing filter sampling ports
2 - slow sand filter influent
2-1, 2-2, 2-3, 2-4, 2-5 - slow sand filter sampling ports
3 - GAC filter influent (pilot 1) or slow sand filter 2 influent (pilot 2)
3-1, 3-2, 3-3 - sampling ports
4 - effluent

3.3 ANALYTICAL METHODS AND QUALITY ASSURANCE/QUALITY CONTROL

3.3.1 Water Sampling Collection Method

Water samples were obtained starting at the most downstream sample point (ie. sample location #4 in Figure 3.4) and progressing backwards to upstream sampling ports throughout the pilot system. This was done to avoid sampling error due to potential disruption of flow caused by sampling, and also minimized cross-contamination of samples when using the same sample collection container for all of the sampling points (ie. sampling ports with the cleanest water were sampled first). Samples that were analyzed onsite were collected in a flask, beaker, or vial, which were rinsed with ultra pure water between samples, and rinsed three times with water from the sampling port before collection of the sample. Samples that were collected for transport back to the university laboratory were collected in a clean container that was rinsed with sample water before collection of the sample.

3.3.2 Turbidity

Turbidity of the raw water and process effluent were monitored using online HACH 1720D turbidimeters. Turbidity at different stages in the treatment process, including raw water and process effluent was also measured using a portable HACH 2100P turbidimeter. The online turbidimeters were calibrated using user-prepared formazin standards of 0 and 20 NTU, which were diluted from a 4000 NTU stock solution. The portable turbidimeter was calibrated using StablCal Stabilized Formazin Standards of <0.1, 20, 100, and 800 NTU, which are the primary standards in Hach Method 8195, an acceptable version of USEPA Method 180.1.

For quality control, the turbidity of pure water (blank sample) was measured daily with the handheld turbidity meter before any sample measurements, and it ranged from 0.03 to 0.08 NTU. Since the blank measurement was less than 0.1 NTU, no correction factor was applied to the sample measurements.

A sampling method triplicate was conducted daily on the effluent of the slow sand filter to determine the sampling error in handheld turbidity measurements. Throughout the study

(Feb.-June), the standard deviation of the triplicate measurements ranged from 0 to 0.081. The instrument error of the handheld turbidity meter was determined by measuring the standard deviation of a triplicate of measurements on the same sample. The resulting standard deviation was 0.02. Thus, there was good precision in the sampling method and instrument analysis in the handheld turbidity measurements.

3.3.3 Temperature

Water temperature was measured onsite with a portable Orion 835 dissolved oxygen/temperature probe meter, which was re-calibrated daily before sample measurements.

3.3.4 Total and Fecal Coliforms

Total coliforms were analyzed according to Method 9221: Multiple Tube Fermentation Technique for Members of the Coliform Group and Method 9020:9 Quality Assurance/Quality Control: Verification in Standard Methods for Water and Wastewater Treatment. The presumptive and confirmed phases of analysis were conducted on all total coliform samples. Positive tubes were scored using the most probable number (MPN) Table 9221.IV in Standard Methods and reported as MPN/100 mL.

Fecal coliforms were analyzed according to Method 9222: Fecal Coliform Membrane Filter Procedure in Standard Methods for Water and Wastewater Treatment. The presumptive, confirmed, and complete phases of analysis were conducted on all fecal coliform samples.

3.4 *CRYPTOSPORIDIUM* CHALLENGE TEST METHODS

The following protocols were initially developed as a part of an American Water Works Association Research Foundation Project conducted at the University of Waterloo. Additional details are provided in Huck et al. (2001 and 2002), Emelko (2001), and Emelko, Huck and Douglas (2003).

The purpose of the *Cryptosporidium* challenge tests was to determine the removal of *Cryptosporidium parvum* (*C. parvum*) in the pilot multistage filters. The challenge tests

employed continuous seeding of a formalin-inactivated *C. parvum* oocyst feedstock into the influent of the slow sand filter in each pilot system. Seeding was continuous for a pre-determined period of time and water samples were collected at the effluent of the slow sand filter and effluent of slow sand filter 2 (in pilot 2) at specific time intervals. According to Emelko (2001), formalin-inactivated *C. parvum* oocysts are reliable surrogates for viable oocysts, thus a representative efficiency of removal can be determined from the challenge tests.

The challenge tests were conducted after a period of stable operation during which multistage filter effluent turbidities were continuously below 5 NTU. During the challenge tests, the raw water pH, dissolved oxygen, temperature, turbidity and slow sand filter effluent turbidity were monitored. These results are shown in Appendix E: Table E.1 to E.3 A summary of all of the *Cryptosporidium* challenge tests is shown in Table 3.5.

Table 3.5: Summary of *Cryptosporidium* challenge tests

Test #	Date	System	Filter Challenged	Hydraulic Loading Rate (m/h)	Sampling Locations
5a	31-May-04	Pilot 1 (Train 1)	SSF	0.4	2*, 3
5b	7-Jun-04	Pilot 1 (Train 1)	SSF	0.8	2*, 3
5c	28-Apr-04	Pilot 2	SSF-1 and SSF-2	0.4	2*, 3, 4

Notes:
SSF - slow sand filter
2* - 5cm above slow sand filter bed
3 - effluent of slow sand filter
4 - effluent of slow sand filter 2 (pilot 2)

3.4.1 Preparation of *Cryptosporidium* Feedstock

The inactivated *C. parvum* oocysts were obtained from a commercial laboratory (Waterborne, Inc., New Orleans, LA) and were bovine in origin. They were provided in a clean, purified form. Two 50 mL vials of 10^9 oocysts, each with a concentration of 20,000,000,000 oocyst/L, were obtained. The oocysts were inactivated with 5% formalin (final concentration) in 1X PBS with 0.01% Tween 20 (J.T. Baker Chemical Co., Philadelphia, PA) to prevent oocyst clumping. Inactivation of oocysts with formalin has been shown to not influence the surface

charge characteristics of the oocysts (Butkus et al., 2003). All *Cryptosporidium* stock solutions were refrigerated at 4°C in the dark until use.

The *C. parvum* oocyst feedstock was prepared by vortexing the 50 mL vial of oocysts and adding 23 mL of the oocysts to 1-L of raw water, resulting in a feedstock concentration of 460,000,000 oocysts/L. The oocysts were added to raw water to equilibrate with the presence of natural organic matter (NOM) in the source water. This enhances the electrostatic repulsion between the oocysts and increases their hydrophobicity (Dai and Hozalski, 2002), which is more representative of actual oocyst behaviour in NOM affected source water.

3.4.2 Seeding Protocol

Prior to, and during seeding, the seed suspension was constantly agitated with a magnetic stirrer to ensure a reasonably homogeneous distribution of the oocysts in the feedstock solution. Prior to seeding, samples were collected from the feedstock to confirm the oocyst feedstock concentration.

A peristaltic pump was used to seed the feedstock into the raw water influent of the multistage filter. The feedstock was introduced into the influent piping approximately 0.6 m before the filter to allow good mixing with the influent. The oocysts were seeded into the influent for a period of 6 hours, at a rate of 1 to 2 mL/min, depending on the flow rate of the raw water influent, resulting in a target influent concentration of 10^6 oocysts/L. For details on the feedstock and seeding parameters, see Table 3.6.

Table 3.6: Feedstock and Seeding Parameters

	Pilot System 1 (0.4 m/h*)	Pilot System 1 (0.8 m/h*)	Pilot System 2 (0.4 m/h*)
Flow of raw water (L/min)	0.499	0.998	0.459
Flow of feedstock (L/min)	0.001	0.002	0.001
Total flow of influent (L/min)	0.5	1	0.46
Conc. of raw water (oocyst/L)	0	0	0
Conc. of feedstock (oocyst/L)	460,000,000	460,000,000	460,000,000
Target Conc. of influent (oocyst/L)	1,000,000	1,000,000	1,000,000
Duration of test (min)	360	360	360
Required volume of feedstock (L)	0.39	0.78	0.36

* filtration rate through slow sand filter

3.4.3 Sampling Protocol

During the *Cryptosporidium* challenge test, water samples were collected at the slow sand filter influent and effluent locations in 250 mL amber bottles and 1-L glass Wheaton bottles, respectively. The sampling locations for each test are detailed in Table 3.5. Aliquots from the oocyst feedstock were collected in 4-mL glass chromatography vials to determine the actual feedstock concentration. Prior to each seeding experiment, 1-L negative controls were collected at the filter influent and effluent locations to determine if any background concentrations of *C. parvum* existed. The samples were stored at 4^{o}C for no longer than four weeks before analysis.

Prior to use, all sampling containers were washed, autoclaved, and rinsed with a few millilitres of a buffered detergent solution to prevent sticking of oocysts to the wall of the container (1× phosphate buffered saline [PBS] with final concentrations of: 0.1% sodium dodecyl sulfate, 0.1% polyoxyethylene sorbitan monooleate [Tween 80, J.T. Baker Chemical Co., Philadelphia, PA], and 0.01% silicone polymer foam depressor [Sigma Antifoam A, Sigma-Aldrich Corp., St. Louis, MO] and final pH of 7.4). The excess surfactant solution was discarded.

3.4.4 Tracer Studies to Determine Sampling Times

Tracer studies were required to characterize the flow profile through each pilot system, determine the seeding time required to obtain steady state influent and effluent concentrations, and quantify the actual hydraulic detention time of each filter. The observed detention time was then used as the lag time between the time of sampling for the influent and the effluent, such that the effluent samples would correspond to the influent samples.

The theoretical hydraulic detention time can be calculated by multiplying the empty bed contact time (EBCT) of the filter by its media porosity. However, this calculation is only representative under ideal conditions. In reality, the detention time of each filter is dependent on the amount of standing water level above the filter bed, which may vary based on the influent turbidity, flow, and headloss across the filter.

Therefore, tracer tests were conducted during a period of stable and typical raw water quality conditions, where raw water turbidity was consistently below 5 NTU and final effluent turbidities were consistently below 0.3 NTU. These were also the conditions in which subsequent *Cryptosporidium* challenge tests were conducted. In addition, the roughing filter and slow sand filter were cleaned at least 7 days prior to both the tracer and *Cryptosporidium* challenge tests to make sure cleaning would not be required during the tests. This also allowed ample time to re-establish the biofilm in the sand filter after cleaning.

A total of three tracer tests were conducted, each simulating the proposed filter operating conditions for each *Cryptosporidium* challenge test. The filter operating conditions for the three tests are shown in Table 3.7.

Table 3.7: Filter Operating Conditions During Tracer Tests and *Cryptosporidium* Challenge Tests

	Pilot 1 - Train 1 (0.4 m/h*)	**Pilot 1 - Train 1 (0.8 m/h*)**	**Pilot 2 (0.4 m/h*)**	
			SFF1	SSF2
Diameter of filter column (mm)	315	315	295	150
Area (m^2)	0.078	0.078	0.068	0.018
Flow (L/min)	0.50	1.00	0.46	0.12
Flow (m^3/hr)	0.030	0.060	0.027	0.007
Filtration rate (m/hr)	0.4	0.8	0.4	0.4
Depth of media** (m)	0.60	0.60	1.15	0.65
Volume of media (m^3)	0.047	0.047	0.079	0.011
Empty Bed Contact Time (hr)	1.56	0.78	2.88	1.63
Hycraulic detention time of media*** (hr)	0.70	0.35	1.29	0.73
Depth between influent sampling port and top of sand (m)	0.05	0.05	0.05	0.05
Volume of water between influent sampling port and top of sand (m^3)	0.004	0.004	0.003	0.001
Hydraulic detention time between influent sampling port and top of sand (hr)	0.13	0.06	0.13	0.13
Total theoretical hydraulic detention time (hr)	0.83	0.42	1.42	0.86

* filtration rate through slow sand filter
** including depth of underdrain gravel
*** assuming media porosity of 0.45
SSF1 - slow sand filter 1
SSF2 - slow sand filter 2

The tracer tests involved the step-dose (continuous) application of a conservative tracer to the filter influent and measurement of the effluent concentration as a function of time. The tracer was applied for a period of at least 2 to 3 times the theoretical detention time of the filter, which was calculated in Table 3.7, to achieve a steady state concentration in the effluent (Hudson, 1981). Chloride was chosen as the optimal tracer, as it is conservative, non-reactive, and unlikely to interfere with biological processes in the filter.

To determine if conductivity measurements could be used to monitor the tracer, a calibration curve was first generated using raw water from the Grand River and a series of increasing standards of chloride concentration. The resulting calibration curve, shown in Figure 3.14,

displays a highly linear correlation between increasing chloride concentration and conductivity, with a coefficient of determination (R^2) of 0.9989. Thus, conductivity measurements were deemed appropriate to monitor the chloride tracer during the tracer tests.

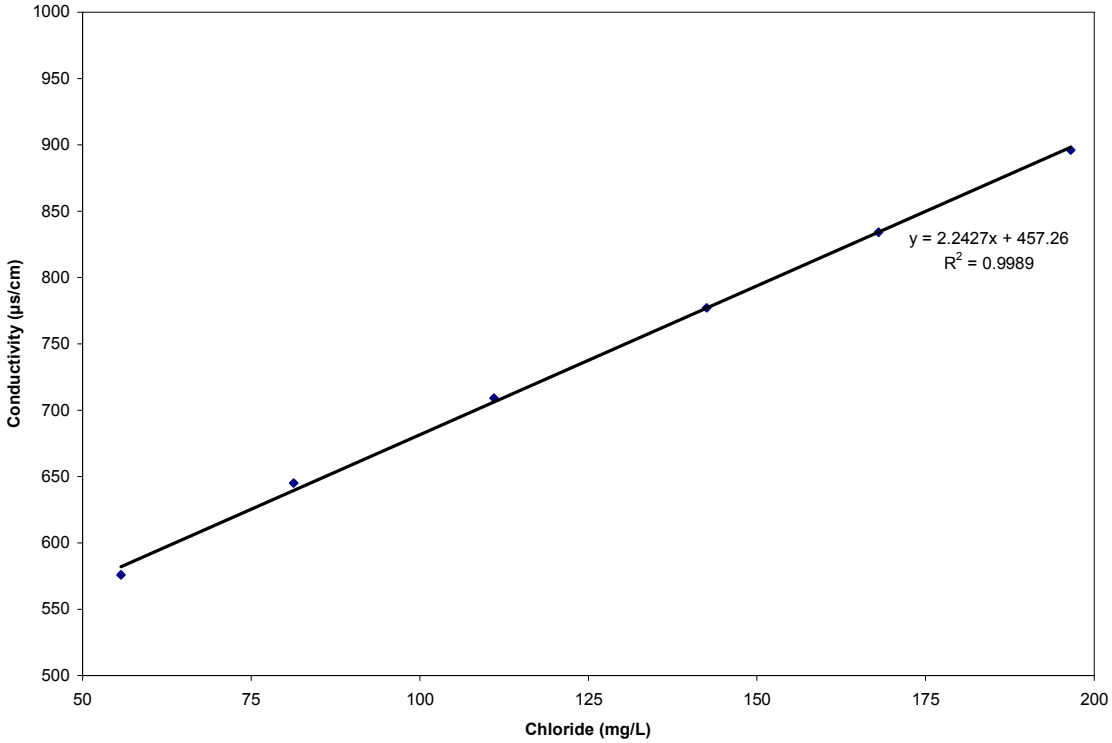

Figure 3.14: Calibration Curve – Chloride vs. Conductivity (Grand River water)

A chloride dose of approximately 110 mg/L (after mixing with raw water) was used in the tracer tests, which was approximately 1.5 times the background chloride concentration of the Grand River. This also correlated to a conductivity of 1.5 times the background conductivity.

The tracer was metered into the influent for 5 to 6 hours and conductivity measurements were taken at 10 to 20 minute intervals. After 5 to 6 hours, the application of tracer was ceased and conductivity measurements were continued until the effluent conductivity returned to the original background conductivity in the Grand River. The results of the tracer tests for pilot 1 (train 1) at 0.4 m/h, pilot 1 (train 1) at 0.8 m/h, and pilot 2 at 0.4 m/h are shown in Figure 3.15, Figure 3.16, and Figure 3.17, respectively. For further details of the sampling locations shown in the legend, see Figure 3.4 and Figure 3.10.

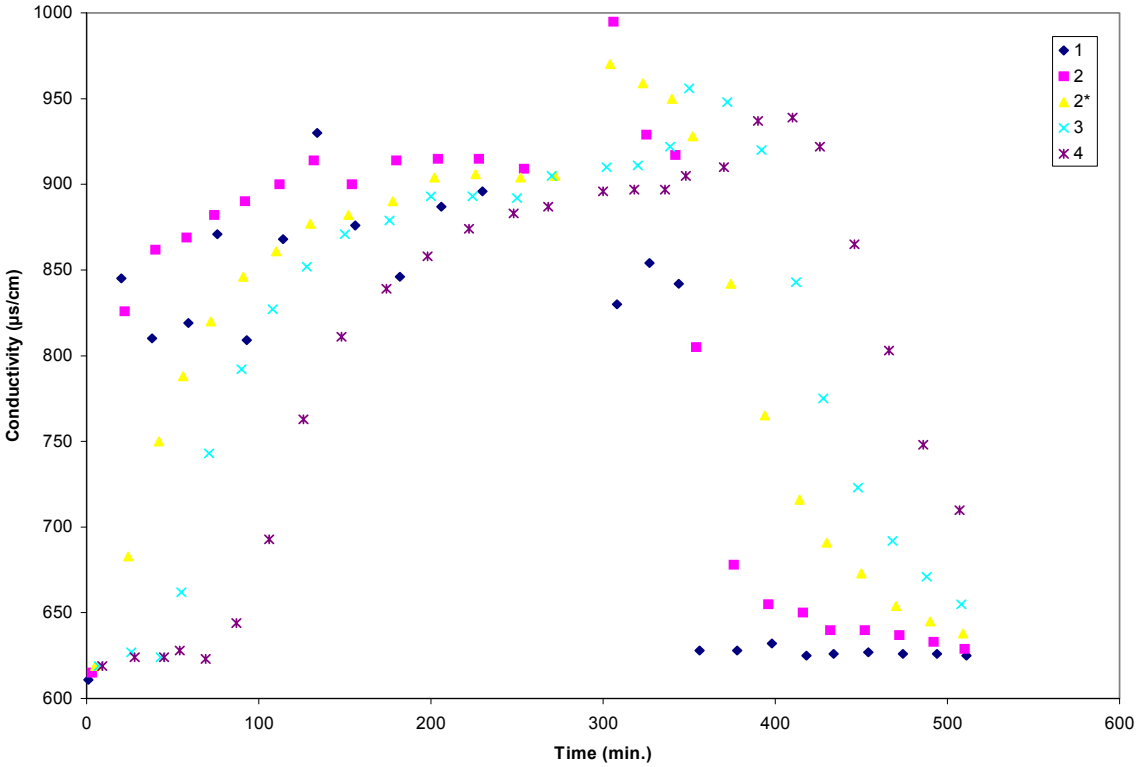

Figure 3.15: Tracer Test Results for Pilot 1 (Train 1) at 0.4 m/h

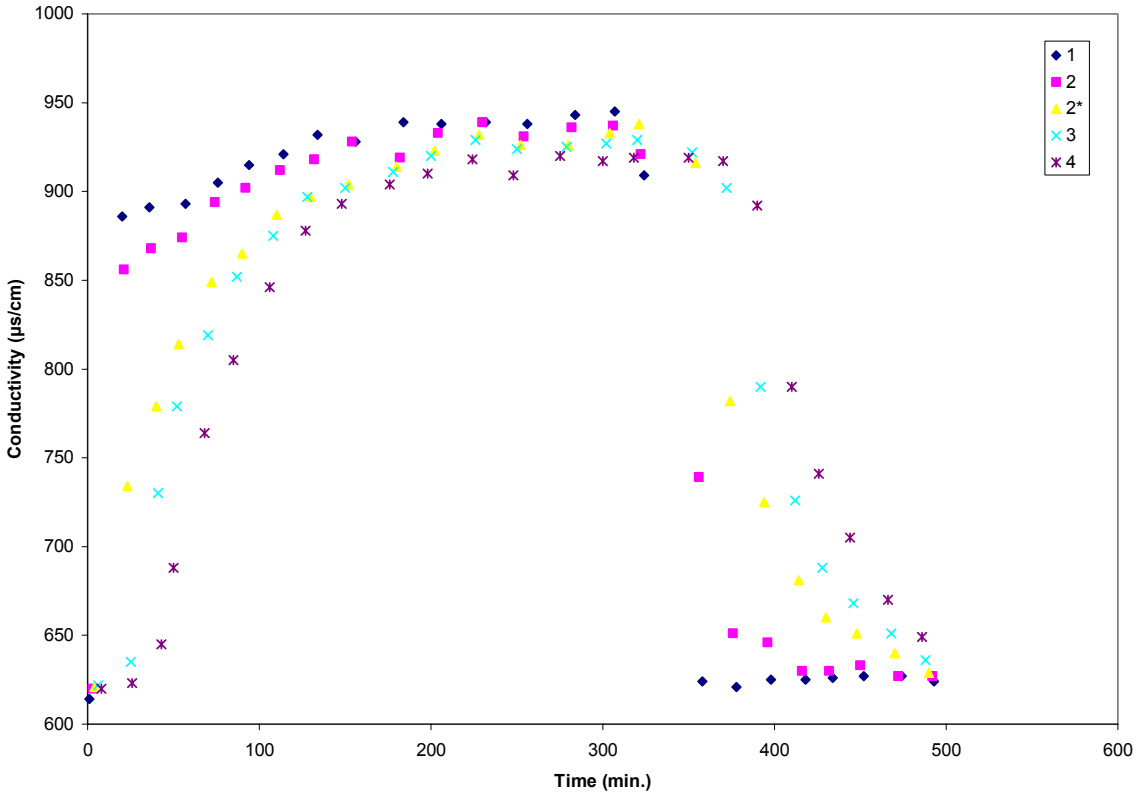

Figure 3.16: Tracer Test Results for Pilot 1 (Train 2) at 0.8 m/h

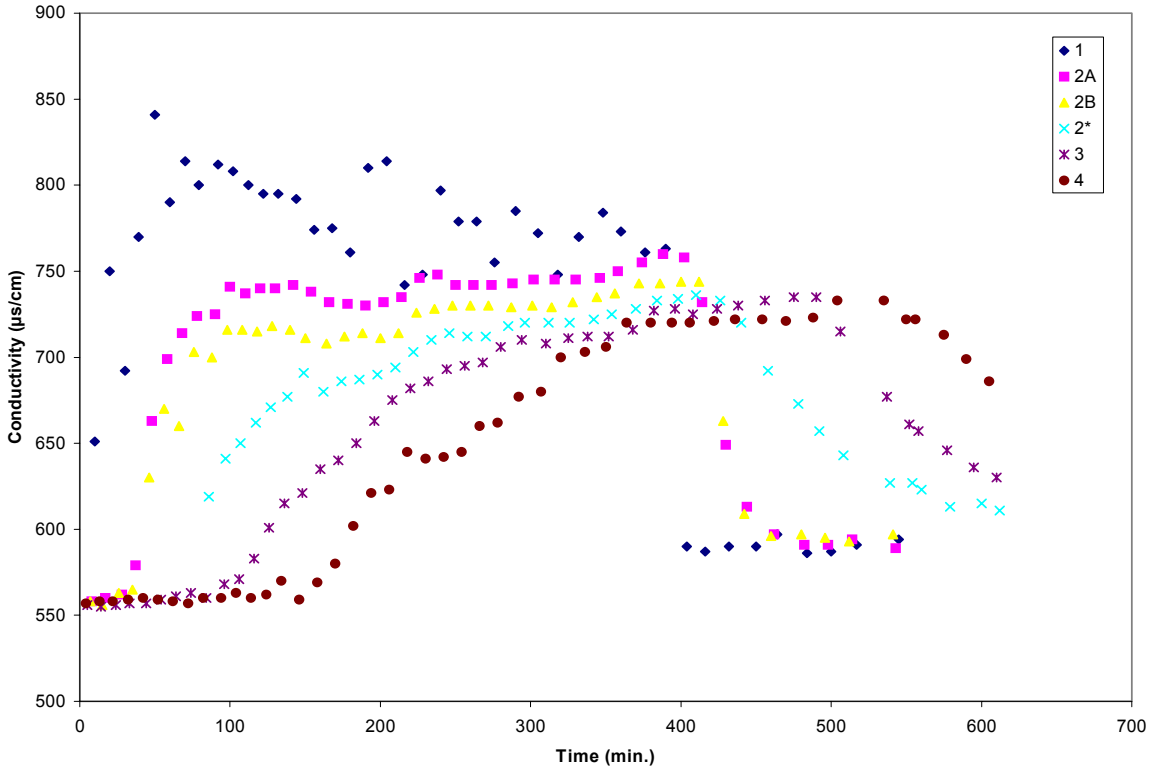

Figure 3.17: Tracer Test Results for Pilot 2 at 0.4 m/h

In the first tracer test (Pilot 1 at 0.4 m/h), Figure 3.15 shows that the roughing filter and slow sand filter effluent conductivity approached steady state (stopped increasing) after approximately 240 and 300 minutes, respectively. Due to a 15-minute failure in the raw water pump that occurred at 260 minutes, a large spike of chloride was injected into the filter influent without adequate dilution with raw water. The raw water pump was re-activated at 285 minutes, which resulted in a roughing filter effluent spike at 300 minutes and slow sand filter effluent spike at 350 minutes. Since the steady state effluent concentration had already been achieved when the spike occurred, this accidental chloride spike was actually useful in determining the actual detention time of the roughing filter and slow sand filter to be approximately 15 minutes and 50 minutes, respectively. Interestingly, the actual detention time for the slow sand filter was close to the theoretical hydraulic detention time calculated in Table 3.7. This is likely because the water level above the sand bed during the tracer test (~7.5 cm) was similar to the water level (or height of the influent sampling port) used in the calculation for theoretical detention time. For simplicity, the assumed hydraulic detention

during time during *Cryptosporidium* challenge test #5a was one hour, such that corresponding influent and effluent samples were obtained 1 hour apart.

In the second tracer test (Pilot 1 at 0.8 m/h), Figure 3.16 shows that the roughing filter and slow sand filter effluent concentration reached steady state at approximately 250 minutes. It is also evident that after the tracer application was terminated at 305 minutes, the roughing filter effluent concentration began to decrease almost immediately and the slow sand filter effluent concentration began to rapidly decrease approximately 45 minutes later. This indicates that the actual hydraulic detention time of the roughing filter was under 10 minutes, and that of the slow sand filter was about 45 minutes.

Although the first two tracer tests operated at different hydraulic loading rates of 0.4 m/h and 0.8 m/h, respectively, the observed hydraulic detention times during both tests were similar. Due to the higher hydraulic loading in the second test, there was much higher headloss across the filter bed, which resulted in a higher standing water level in the filter (~60 cm above the sand). Thus, due to the large volume of standing water, which had a maximum steady state concentration at the time tracer application was terminated, the time it took for the effluent concentration to decrease from steady state was much longer than the theoretical detention time calculated in Table 3.7. However, during the *Cryptosporidium* challenge tests, influent samples were not collected from the top of the standing water in the filter, rather they were collected just 5 cm above the sand bed, thus the actual detention time from this point to the effluent would be that which was calculated in Table 3.7 (approximately 25 min.). Thus, *Cryptosporidium* challenge test #5b assumed a hydraulic detention of 30 minutes, such that corresponding influent and effluent samples were obtained 30 minutes apart.

In the third and final tracer test (Pilot 2 at 0.4 m/h), Figure 3.17 shows that the roughing filter, slow sand filter 1, and slow sand filter 2 effluent concentration reached steady state at approximately 120, 360, and 400 minutes, respectively. It is also evident that after the tracer application was terminated at 390 minutes, the roughing filter, slow sand filter 1, and slow sand filter 2 effluent concentrations began to decrease at 410, 490, and 535 minutes, respectively. This suggests that the actual hydraulic detention time of the roughing filter,

slow sand filter 1, and slow sand filter 2 was approximately 20, 80, and 45 minutes, respectively, which are similar to the theoretical hydraulic detention times calculated in Table 3.7. However, since test #5c was actually the first *Cryptosporidium* challenge test conducted, a conservative and simplified detention time of 2 hours was assumed for the slow sand filter 1, and 1 hour was assumed for slow sand filter 2. Thus, corresponding influent and slow sand filter 1 effluent samples were collected 2 hours apart, whereas corresponding influent and slow sand filter 2 effluent samples were collected 3 hours apart.

Overall, from the tracer tests it was determined that during the *Cryptosporidium* challenge tests, the *Cryptosporidium* feedstock would be seeded to the filter influent for a period of 6 hours. This would ensure that a steady state filter effluent concentration would be reached before feedstock seeding was terminated. To quantify the removal of *Cryptosporidium* throughout the duration of each test, influent and effluent sampling times were offset by 1 hour for test #5a, 0.5 hour for test #5b, and 2 hours for test #5c, to obtain corresponding influent and effluent samples.

3.4.5 Calculation of Microorganism Concentration and Removal

C. parvum oocyst removals ($\log_{10}$) were calculated by subtracting the log of the filter effluent concentration from the log of the influent concentration. When no oocysts were detected, the concentration was reported as 0, however, removal was calculated by using a concentration of 1 oocyst/sample volume processed and normalized to 1 L. For example, a value of 1 oocyst/L would be used in the log removal calculation if no oocysts were found in a 1-L sample; a value of 2 oocysts/L would be used in the calculation if no oocysts were found in a 500-mL sample.

3.4.6 Enumeration of Feedstock

Prior to each challenge test, an aliquot from the oocyst feedstock was collected in a 4-mL glass vial, brought back to the University of Waterloo laboratory, was briefly vortexed, and a small portion of the suspension (<100 µL in total) was removed to enumerate the oocyst concentration. The feedstock concentration was determined by averaging 3 to 5 replicate

counts with a hemacytometer (Petroff-Hausser Bacterial Counting Chamber, Hausser Scientific Corporation, Horsham, PA) and light microscopy (Ziess Axioscope 2, Empix Imaging, Mississauga, ON). The entire hemacytometer grid (1 mm^2) was used in the oocyst enumeration process. The resulting average feedstock concentrations ranged approximately from 415,000,000 to 460,000,000 oocysts/L with a standard deviation ranging approximately from 29,000,000 to 230,000,000 (see Appendix E: Table E.4). These results were very similar to the calculated target feedstock concentration of 460,000,000 oocysts/L, shown in Table 3.6.

3.4.7 Analytical Protocol

Filter influent samples were analyzed in 1 to 2 mL volumes and filter effluent samples were analyzed in volumes ranging from 200 to 500 mL. These sample volumes were generally chosen to yield between 20 and 1000 oocysts per membrane. All pipettes, glassware, and apparatus in direct contact with the sample were pre-rinsed with the buffered detergent solution to prevent oocyst losses.

All samples were filtered through 25-mm, 0.40 µm polycarbonate membranes (Nucleopore, Corning, Acton, MA). The filter membranes were placed on top of 25-mm, 8.0 µm nitrocellulose support membranes (MF Millipore Membrane Filters, Millipore Canada, Ltd., Nepean, ON) that were placed on a vacuum filtration manifold (Hoefer Scientific, San Francisco, CA) maintained at a vacuum pressure of 125 mm of mercury. Weights held the membranes in place, and small PVC columns attached to the weights were used to contain the sample water while it was filtering through the membranes.

Two millilitres of 1% bovine serum albumen (BSA) was first passed through the membranes. The samples were then filtered directly on the manifold. The weights and columns that contained the samples during filtering were then rinsed with the buffered detergent solution. The detergent rinse was followed by an additional 2 mL of BSA that was also filtered through the membranes. This procedure was followed by the immunofluorescence assay (IFA)

described below. If necessary, the membranes were kept wet with 1X PBS and covered until sample mounting on slides.

All *C. parvum* oocyst identification was performed with IFA staining techniques (USEPA, 1996), using the Hydrofluor combination *Cryptosporidium* and *Giardia* Kit (Strategic Diagnostics, Newark, DE). After staining, the slides were enumerated using light microscopy in the University of Waterloo laboratory. To ensure accuracy of the enumeration process, approximately every tenth slide was double-counted by a second party. The resulting *C. parvum* concentration was calculated by dividing the number of enumerated oocysts by the sample volume processed.

Recovery data from previous studies using filter influent and effluent matrixes in the University of Waterloo laboratory have shown that the mean oocyst recovery by this procedure was 75% (relative standard deviation was 16%). A statistical method for adjusting concentration and log removal data for analytical recovery was not used, as the recovery for the influent and effluent samples were assumed to be similar. Thus, influent and effluent recoveries would cancel out in the calculation for log removal of *Cryptosporidium*.

3.4.8 Quality Assurance/Quality Control

Sample handling, identification, preservation, transportation, and storage were done according to the USEPA ICR Methods for Protozoa Analysis (USEPA, 1996). The sample site was 15 minutes from the University of Waterloo laboratory, where the samples were analyzed. The samples were processed immediately or refrigerated as specified in the ICR methodology.

The quality assurance/quality control (QA/QC) program included the analysis of *C. parvum* feedstock samples (discussed in Section 3.4.6), negative controls, method blanks, positive controls, and double counting approximately every 10^{th} slide by a second party. Negative controls (samples with no known additions of *C. parvum*) were collected to determine whether any background concentration of *C. parvum* existed. Any background concentration of oocysts in the influent would not affect the challenge test as they would likely be

outnumbered by the large spike of 10^6 oocysts/L. Filter influent and effluent negative controls were collected in 1-L sample bottles, and analyzed in sample volumes ranging from 200 to 400 mL. The results are shown in Table 3.8.

Table 3.8: Concentration of *Cryptosporidium* in Negative Control Samples

Test #	Hour	Concentration (oocyst/L)	
		Influent	Effluent
5c	0	170	0
	1	115,500	0
	2	162,000	0
5a	0	11	208
5b	0	12,490	30

The first challenge test that was conducted was test #5c and influent/effluent samples were collected before the test was initiated (zero hour), at one hour into the test, and at two hours into the test. Since the hydraulic detention time of the slow sand filter was estimated to be over an hour, the first two samples collected were expected to be negative for the presence of oocysts. In fact, all three samples were successful negative controls with zero oocysts detected. The negative control samples for test #5a and test #5b were not as successful, where 208 oocysts/L and 30 oocysts/L were detected in the effluent. The cause of this was unknown and could have been due to error in the method or error in the microscopic enumeration of the slides.

Potential error in method was acquitted by considering the three successful negative controls in test #5c. These were essentially a triplicate of a method blank, where a sample of no known concentration of oocysts was processed in an identical manner to the positive *C. parvum* samples, including processing with all of the reagents used in the processing of *C. parvum*. A method blank ensures that there is no contamination of the water samples with exterior sources of oocyst during sample processing. Since no oocysts were detected in each of the negative controls in test #5c, it can be assumed that there was no contamination with exterior sources of oocysts during sample processing.

Potential error in the microscopic enumeration of the slides can be determined by comparing the oocyst counts of both parties involved in the counting (see Table 3.9). The difference between the counting results between both parties ranged from 8 to 104. The largest difference (104) only impacted the calculated log removal by 0.22 logs (ie. 2.97 logs compared to 2.75 logs). In addition, the average of the difference between the counting results was only 1.25. Thus, the error in the counting results was minimal, and the impact on calculated log removals was minimal. Further, it could not be implicated as the cause of the detection of oocysts in negative controls from test #5a and 5b. Potential causes of this are still unknown.

Table 3.9: Comparison of Microscopic Enumeration Results from Both Counting Parties

Test #	Sample[†]	Count 1	Count 2	Average count	Difference	Standard Deviation
5b	2* - 3	47	67	57	20	14.1
5c	2* - 6	252	333	292.5	81	57.3
5c	3 - 6	73	81	77	8	5.7
5c	3 - 8	260	156	208	-104	73.5
				average	1.25	37.7

[†] 2* - 3 = sampling location - hour of sampling

The final measure in the QA/QC program included the processing of positive control samples. The positive control was used to determine if the method employed was reliable for staining. Well-stained oocysts and cysts are expected to be found in the positive control. If stained oocysts or cysts are not found, the *C. parvum* samples that were processed at the same time would be considered inconclusive due to inadequate staining. Positive control samples were prepared by filtering a 50-µL sample of formalinized stool containing *Cryptosporidium* oocysts and *Giardia* cysts. It was processed in an identical manner to the *C. parvum* samples, including processing with all of the reagents used in the processing of *C. parvum*. In all positive control samples processed, oocysts and cysts were always detected. Thus, the method employed was reliable for producing well-stained countable results.

Chapter 4

4 Background on Grand River Water Quality

4.1 INTRODUCTION

The Grand River Watershed is located in Southwestern Ontario, Canada. It is the "largest Canadian river draining into Lake Erie, with a drainage area greater than 5000 km^{2}" (Droppo and Ongley, 1994). It is the largest watershed in Southern Ontario, is populated by about 800,000 people, and has a number of tributaries such as the Conestoga, Nith, Speed, and Eramosa rivers that pass through several counties and a number of municipalities. "81% of the population lives in urban areas, which represents only 7 percent of the watershed land" (LeChevallier et al., 2002), thus the urban impacts on the river are heavily concentrated in the middle of the watershed, where most of the cities are located. Furthermore, it is one of the largest growing watersheds, in terms of human population, and is expected to "grow 37% over the next 20 years, an increase of 300,000" (GRCA, 2004). Agriculturally active rural areas comprise about 90% of the watershed area (GRCA, 2004), and the river is also heavily impacted by urban runoff, industrial discharge, and wastewater treatment plant effluent. Of the 27 wastewater utilities discharging secondary treatment sewage into the river, 9 of them are located upstream of the Mannheim Water Treatment Plant intake, which is the location of the multistage filter pilot that was evaluated in this study.

The Waterloo Region, which consists of 3 major cities (Waterloo, Kitchener, and Cambridge) and is the major centre of urban activity in the watershed, receives approximately 900 mm of precipitation every year, 765 mm of which is in the form of rainfall (Environment Canada, 2004). During precipitation events, the river experiences extreme fluctuations in turbidity, as well as during the snow melt in the spring, where runoff from agricultural fields causes water levels to rise in the river, resulting in the scour and transport of a large amount of suspended sediment downstream. In fact, LeChevallier et al. (2002) reported a maximum turbidity of 500 NTU in the Grand River.

4.2 GENERAL WATER QUALITY PARAMETERS

A general overview of various water quality parameters for the Grand River is given in Table 4.1, which provides an average of monthly measurements taken over 2 years from 1999 to 2001 by the Provincial Water Quality Monitoring Network (Ontario Ministry of Environment).

Table 4.1: General Water Quality of the Grand River
(adapted from the Provincial Water Quality Monitoring Network (MOE))

Parameter	Units	Average	Standard Deviation
Alkalinity	mg/L	194	36
Aluminum	µg/L	91	88
Ammonium	mg/L	0.6	0.5
Cadmium	µg/L	0.1	0.2
Calcium	mg/L	67	12
Chloride	mg/L	79	25
Chromium	µg/L	0.3	0.6
Conductivity (25^{o}C)	µS/cm	699	119
Copper	µg/L	2.1	0.7
Hardness (total)	mg/L	251	41
Iron	µg/L	143	99
Lead	µg/L	2.1	3.7
Magnesium	mg/L	20	3
Manganese	µg/L	28	16
Nitrates	mg/L	4	1.5
Nitrogen	mg/L	1.3	0.6
pH	-	8.3	0.1
Phosphate	mg/L	0.02	0.01
Phosphorus	mg/L	0.08	0.03
Strontium	µg/L	368	113
Titanium	µg/L	1.5	1.9
Turbidity	NTU	7.4	8.1
Zinc	µg/L	6.1	3.3

According to data collected through the Supervisory Control and Data Acquisition (SCADA) system at the Regional Municipality of Waterloo's Mannheim Intake, the water quality of the Grand River is highly variable throughout the year, likely due to a combination of agricultural, urban, industrial, and seasonal impacts. Measurements of dissolved oxygen, temperature, turbidity, pH, conductivity, and ammonia were averaged daily over a two to three year period from year 2000 to 2003, and are shown in Figures 4.1 through 4.6. The data

used to generate these graphs can be provided by the author on request, as the data set is too large for the Appendix.

In Figure 4.1, the dissolved oxygen levels in the Grand River were highly variable, averaging 2.3 mg/L, and ranging from 0 to 11.3 mg/L (standard deviation of 2.9). It is important to note, however, the period of very low dissolved oxygen from Nov. 2001 to Aug. 2002. It is unlikely that the river would experience such a prolonged period of low dissolved oxygen, thus instrument error was suspected during this time. An analysis of the data set, excluding the data from Nov. 2001 to Aug. 2002, results in an average dissolved oxygen level of 4.25 mg/L (standard deviation of 2.9). The results still indicate a highly variable dissolved oxygen level throughout the year, and periods of dissolved oxygen level below 2 mg/L, which may affect the biological treatment efficiency of a slow sand filter.

The raw water temperature, shown in Figure 4.2, varied seasonally from 3 to 30^{o}C. This large range of temperature is typical of many surface waters in northern climates, and poses a challenge for treatment with traditional slow sand filtration, of which performance relies largely on biological treatment efficiency.

The raw water turbidity, shown as a semi-log plot in Figure 4.3, was highly variable throughout the year, likely influenced by human activities (urban and agricultural) combined with precipitation events. The average turbidity was 4.2 NTU with a maximum of 58 NTU. Since these values are daily averages, the river may have actually experienced higher turbidities throughout the day. In fact, LeChavallier et al. (2002) reported much larger measurements of turbidity, averaging 30.1 NTU with a maximum of 500 NTU.

The pH in the river, shown in Figure 4.4, was also highly variable, averaging pH 7.8 (standard deviation of 1) and reaching as high as pH 8.9. This analysis excludes periods from when the instrument read pH 2, such as in July 2001 and May/June 2002, as this is highly unlikely for a large volume river such as the Grand River (mean flow of 1258 cfs (LeChavallier et al. (2002)), and was probably due to instrument error. Although the pH was usually just over pH 8, there were a few events where the pH experienced sudden drops to below pH 6, possibly

due to industrial discharge or toxic spills. This would indicate the susceptibility of the Grand River to industrial impacts.

Finally, in Figure 4.5, the conductivity of the river averaged 467 µS/cm (standard deviation of 141) and in Figure 4.6, the ammonia concentration averaged 0.4 mg/L (standard deviation of 1). On occasion, the ammonia concentration increased to above 1 mg/L and in some cases reached as high 5 mg/L. This may have been due to wastewater treatment plant discharge upstream of the Mannheim intake. Currently, nine of the twenty-six sewage treatment plants on the Grand River are located upstream of the Mannheim intake.

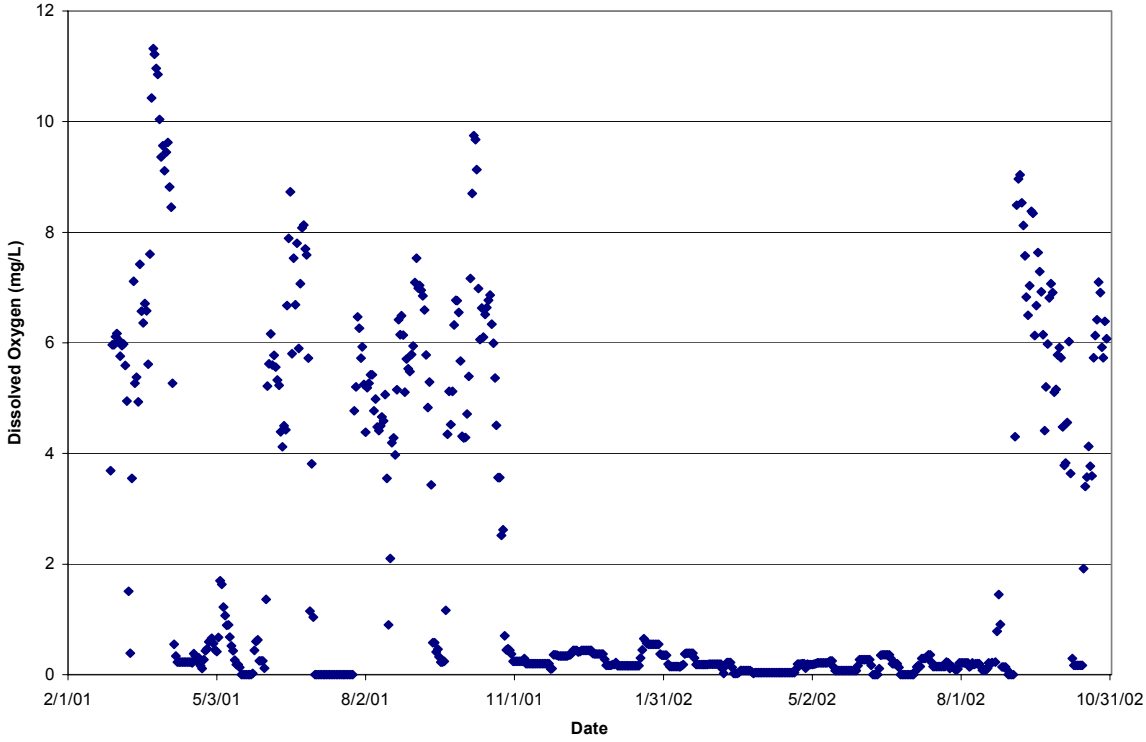

Figure 4.1: Dissolved Oxygen Levels of Grand River at Mannheim Intake (2001-2002)

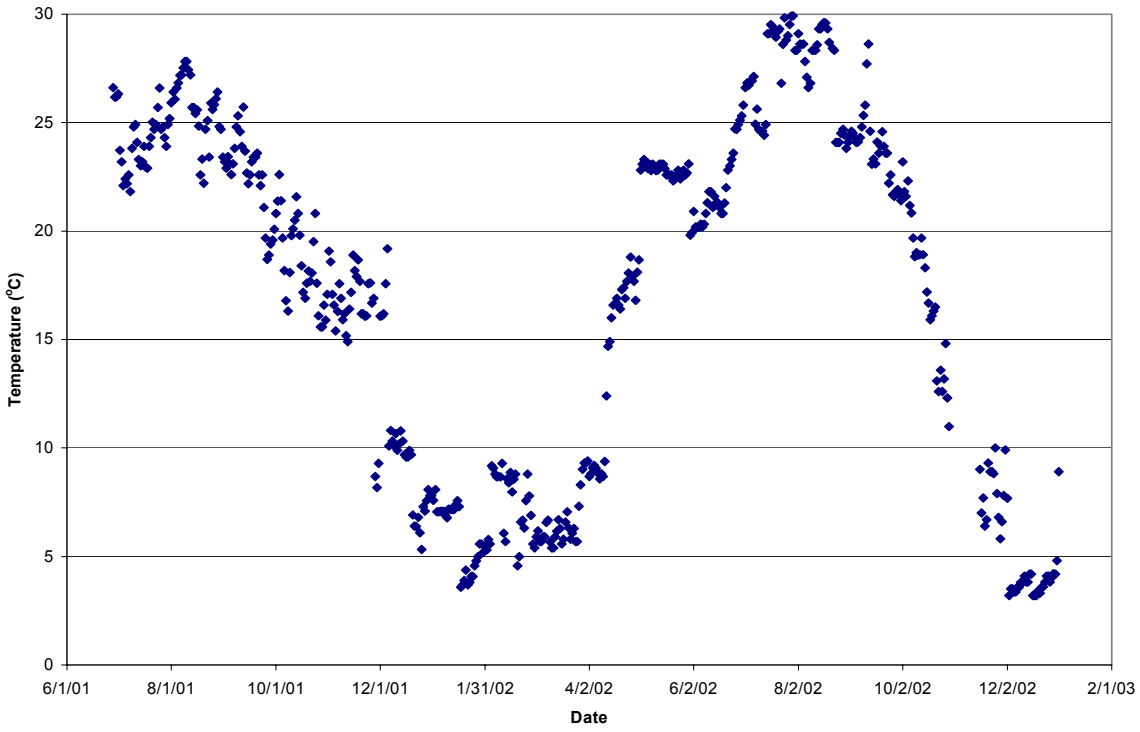

Figure 4.2: Temperature of the Grand River at Mannheim Intake (2001-2003)

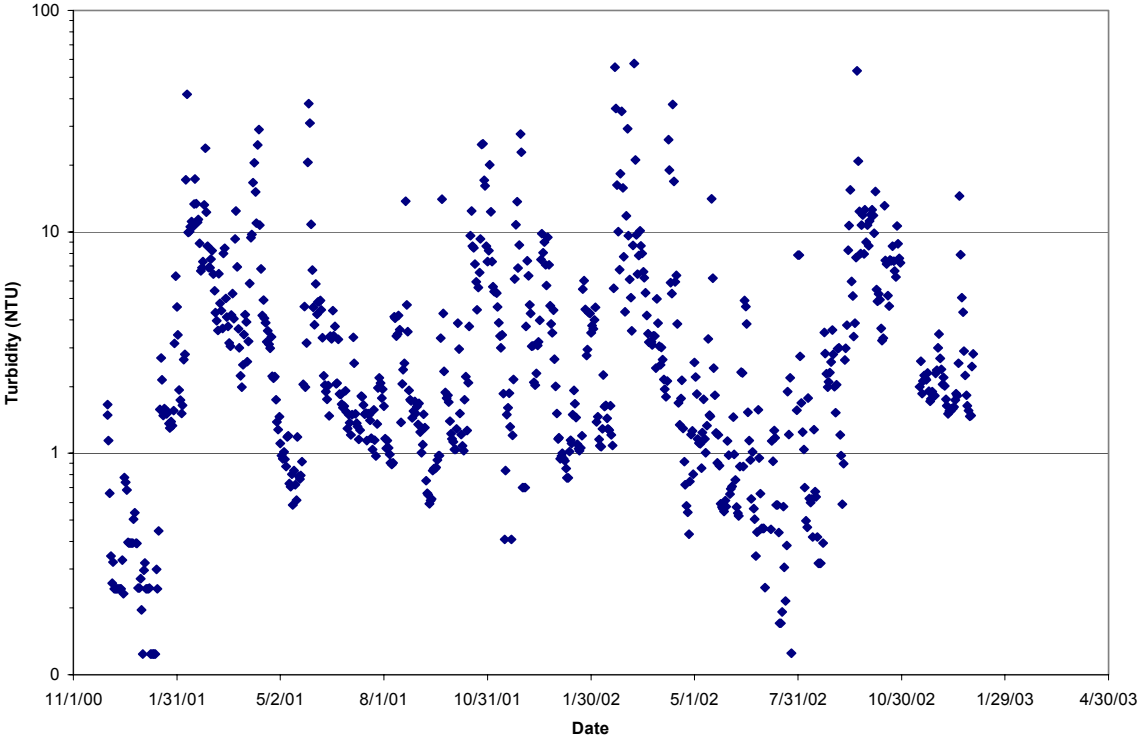

Figure 4.3: Turbidity of the Grand River at Mannheim Intake (2000-2003)

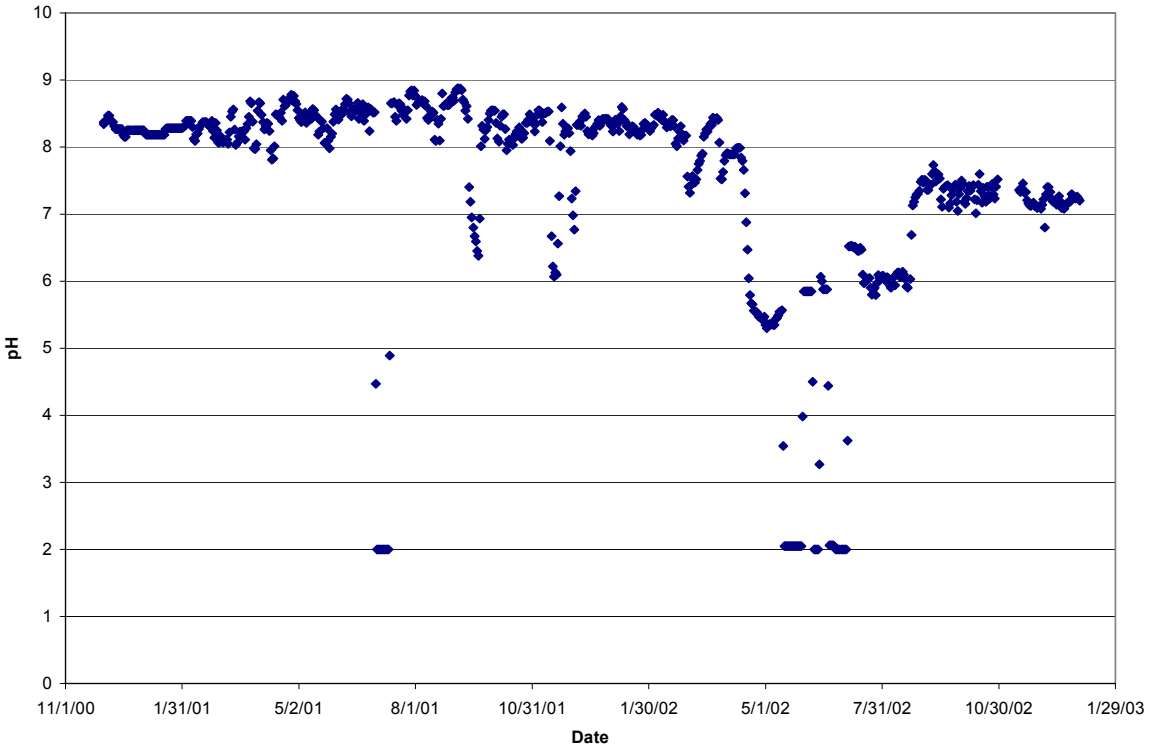

Figure 4.4: pH levels of the Grand River at Mannheim Intake (2000-2003)

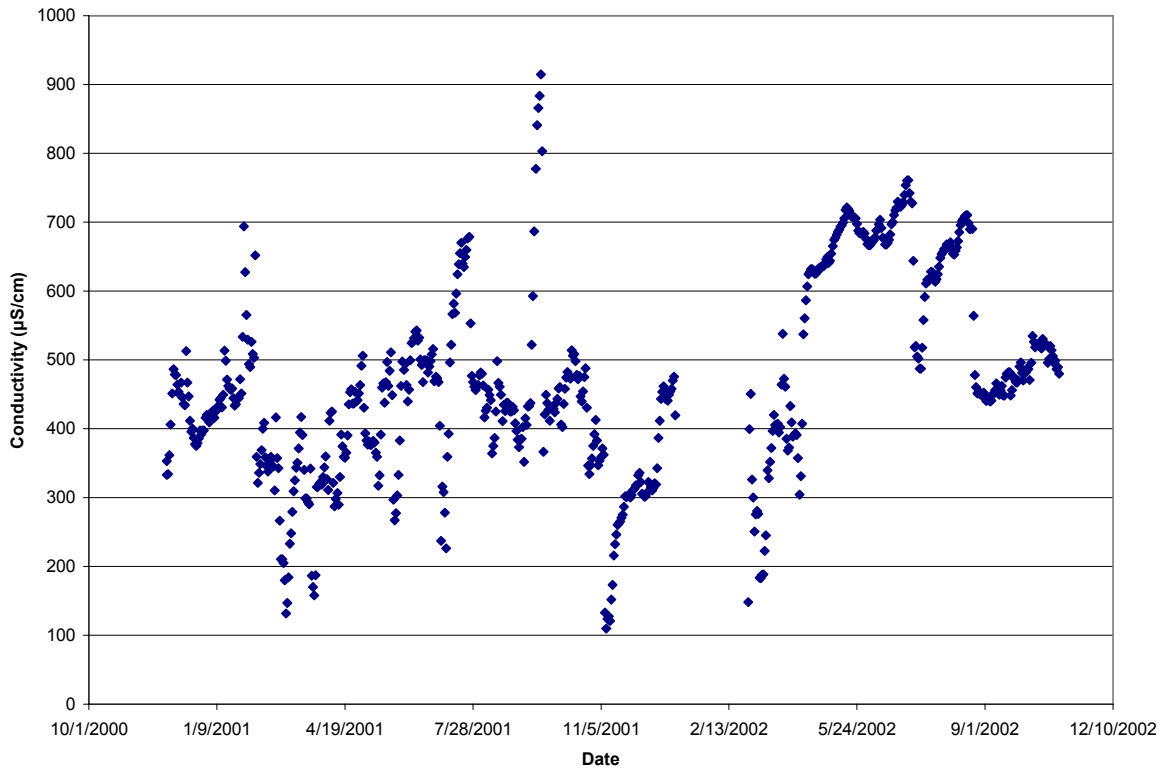

Figure 4.5: Conductivity of the Grand River at Mannheim Intake (2000-2002)

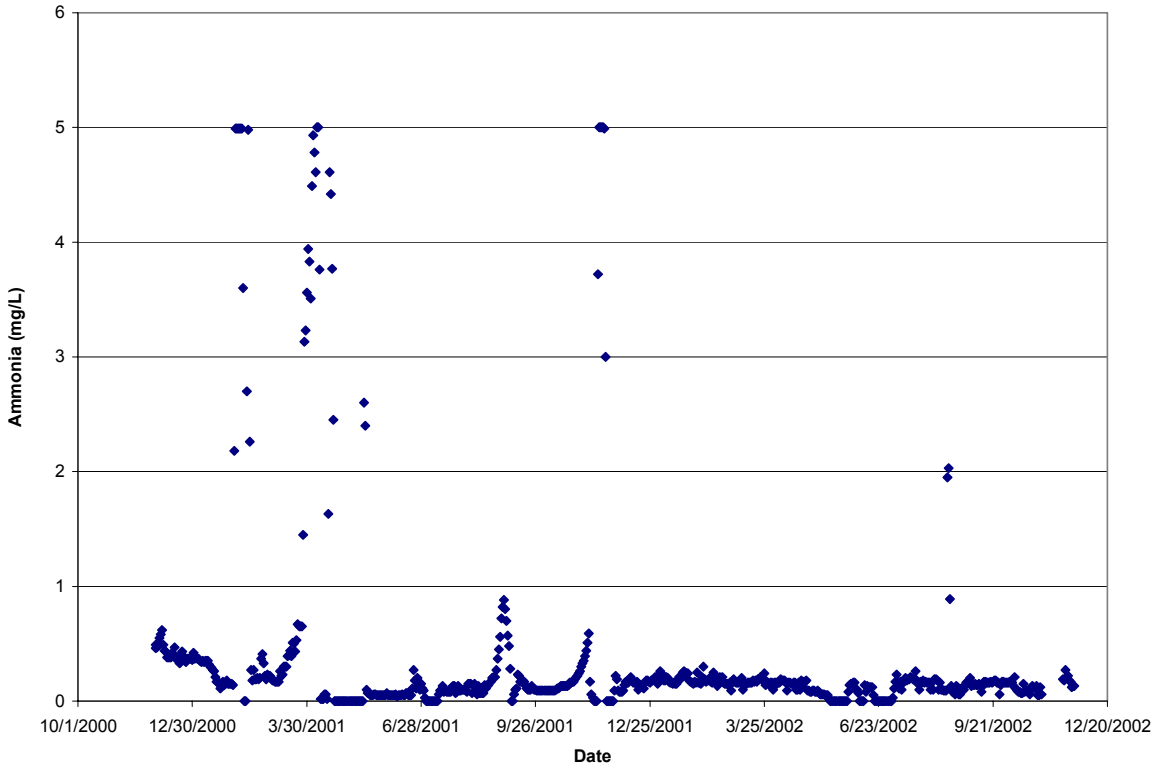

Figure 4.6: Ammonia Concentration of the Grand River at Mannheim Intake (2000-2002)

4.3 SUSPENDED SEDIMENT

The Grand River collects drainage from a variety of soil types, ranging from sand to clay (Ongley, 1974), which are major sources of the suspended sediment (flocs) that are present in the river. A floc is referred to as any particle composed of two or more primary inorganic particles (Droppo and Ongley, 1994). Floc size is relevant because flocs always represent more than 90% of the total volume of sediment transported in a river (Droppo and Ongley, 1994). The average floc size of the Grand River is 9.1 µm and ranges from 2.8 to 219.5 µm, however the majority of flocs are in the relatively small size classes below about 20 µm (Droppo and Ongley, 1994).

Particle size data was obtained on several days throughout the year 2001 by the Provincial Water Quality Monitoring Network (Ontario Ministry of Environment). Samples were collected from the Grand River at Dunnville, Ontario, which is the furthest downstream sampling location in the river before entering Lake Erie. The data suggests that about 90% of

the suspended particles in the Grand River are less than approximately 20 μm in size (see Table 4.2).

Approximately 1% of particles are less than 0.21 μm, however information should be regarded with caution as the lower detection limit of the measurement method was 0.2 μm. Nevertheless, colloidal particles are typically less than 0.1 μm in size (Droste, 1997), thus the majority of suspended particulate matter in the Grand River is not colloidal in nature. This is important because colloidal particles are difficult to remove in water treatment operations without the use of chemicals. Surface charge, which is negative for most particles in water, becomes more important for smaller particles.

However, variations in the particle size distribution of surface water can be expected at certain times throughout the year, depending on the impact of human activities and seasonal variations in precipitation. Therefore, there may be times where a higher proportion of colloidal matter is present in the river, but generally, it appears that colloidal matter is not the primary particulate matter of concern when treating water from the Grand River.

Table 4.2: Particle Size Distribution of Grand River Water at Dunnville, Ontario
(adapted from the Provincial Water Quality Monitoring Network (MOE))

Particle Size (μm)	Proportion (%)	Cumulative (%)
< 0.21	1.0	1.0
0.21 - 0.34	1.7	2.7
0.34 - 0.43	1.3	4.0
0.43 - 0.66	3.3	7.2
0.66 - 1.01	4.7	11.9
1.01 - 1.69	8.4	20.3
1.69 - 2.63	10.4	30.7
2.63 - 3.73	10.4	41.2
3.73 - 5.27	11.8	53.0
5.27 - 7.46	12.3	65.3
7.46 - 10.55	11.1	76.4
10.55 - 14.92	8.8	85.2
14.92 - 21.1	6.0	91.1
21.1 - 29.85	4.2	95.3
29.85 - 42.21	2.8	98.1
42.21 - 62	1.0	99.1

4.4 ORGANIC COMPOSITION

According to Stephenson et al. (1979), who studied the organic composition of Grand River water, "water samples from this river contained dilute concentrations of organic substances originating from natural sources, and a number of industrial and domestic wastewater discharges". They also reported total organic carbon (TOC) levels ranging from 3.15 to 6.69 mg/L at 20^{o}C with an average of 4.96 mg/L (these levels are similar to those found in this study), and the ratio of chemical oxygen demand (COD) to TOC ranging from 1.87 to 2.92. The authors state that this variation in COD/TOC ratio, which indicates the extent to which organics are oxidized, may indicate that the nature of organics in the river fluctuates, likely influenced by human activities in the watershed. In fact, in further ozonation studies, they found that there was no clear optimal ozone dose for water from Grand River, possibly due to the changing nature of the NOM.

Samples that were collected throughout the course of the present study indicate an average SUVA (specific ultraviolet absorbance) of 2.98 L/mg-m, which indicates a more non-humic, hydrophilic and lower molecular weight organic matter that is generally more easily biodegradable.

4.5 *GIARDIA*, *CRYPTOSPORIDIUM*, AND BACTERIA

According to LeChevallier et al. (2000), *Giardia* and *Cryptosporidium* were detected in 51.6% and 35.5% of Grand River samples (approximately 100 samples), respectively. In LeChavallier et al. (2002), *Giardia* concentrations in the river averaged 30.5 cysts/L with a maximum of 158 cysts/L, while *Cryptosporidium* concentrations averaged 43.7 oocysts/L with a maximum of 367 oocyst/L.

In the study conducted by Anderson et al. (2003), fecal coliforms were present in 95 to 100% of Grand River watershed samples (total of 320 samples from approximately 50 sample locations) and *E. coli* was present in 91 to 100% (*E. coli* O157:H7 was present in 6 to 7%).

LeChavallier et al. (2000 and 2002) quantified the concentrations of total and fecal coliforms in the Grand River, where average total coliform levels ranged from 2,600 to 58,911 cfu/100

mL with a maximum of 9,700,000 cfu/100 mL, while average fecal coliform levels ranged from 433 to 540 cfu/100 mL with a maximum of 20,000 cfu/100 mL. The total number of samples and sample locations were not given but it would appear that it was approximately 40 to 100 samples.

4.6 SUMMARY

Overall, the water quality of Grand River is heavily impacted by agricultural, urban, and industrial activities. Water quality parameters such as turbidity, dissolved oxygen, temperature, and pH are highly variable throughout the year. In addition, large fluctuations in total organic carbon levels and the changing nature of natural organic matter indicate that the watershed is greatly influenced by human activities.

Particle size data indicates that the majority of suspended sediment in the river is less than 20 μm in size, thus posing a challenge from a filtration point of view, however the presence of stable suspended colloidal matter is generally not a concern.

In addition, the relatively high prevalence of *Giardia* and *Cryptosporidium* in the river and the consistent presence of coliform bacteria and *E. coli*, usually in high levels, pose a threat for potential waterborne disease outbreaks if drinking water obtained from the river is not reliably treated.

Therefore, it is evident that the Grand River is a challenging source water to treat for conventional treatment facilities, and a particularly interesting challenge for the non-chemically assisted multistage filter system that will be evaluated in this study. The fact that the multistage filter system does not require chemical coagulants, of which dosage is so dependent on source water chemistry, will be advantageous from an operation and maintenance standpoint, especially because the Grand River has such a variable water quality and chemistry.

It will be interesting to observe the performance of the multistage system during such fluctuations in source water quality, with little or no process adjustment, and determine

whether it will prove to be a safe and robust treatment alternative for challenging surface waters such as the Grand River.

Chapter 5

5 Performance of Multistage Filtration: Results from Start-up Turbidity, Online Turbidity, and Operational Headloss Data

5.1 INTRODUCTION AND OBJECTIVES

As indicated previously, the primary objective of this study was to investigate the potential of multistage filtration technology as a reliable and robust drinking water treatment process to treat surface water for small and non-municipal water systems in northern climates, where seasonal fluctuations in surface water turbidity and temperature pose challenging conditions from a treatment standpoint. The detailed objectives for the portion of the study described in this chapter were to determine:

1. The performance of multistage filtration in removing turbidity during the acclimatization period with colder water temperatures,
2. The efficiency of multistage filtration in removing turbidity with variable raw water turbidity levels (up to and above 100 NTU) and influent water temperatures (3^{o}C to 20^{o}C),
3. The effect of hydraulic loading rate (0.2 to 0.8 m/h) on effluent turbidity,
4. The impact of pre-ozonation and post-granular activated carbon stages on removal of turbidity,
5. The impact and added robustness of an additional stage of slow sand filtration,
6. The impact of operational disturbances, such as sudden increases in hydraulic loading rate and filter cleaning, on effluent turbidity, and
7. The factors affecting filter run length.

5.2 MATERIALS AND METHODS

5.2.1 Pilot Facilities

As discussed previously, this study involved the testing and comparison of two pilot multistage filtration systems. Pilot system 1, constructed by MS Filter Inc., consisted of two identical treatment trains. A detailed process diagram was given in Figure 3.4. Pilot system 1 featured shallower bed depths (due in part to size constraints in the trailer in which it is located) and smaller roughing filter media than recommended in the literature. In addition, the roughing filter featured a novel modification, the use of granular activated carbon (GAC) for the fine media layer. Furthermore, it featured a pre-ozonation stage prior to the roughing filter, and a GAC filter following the slow sand filter mainly for removal of organics and colour. A detailed description of pilot system 1 was given in Section 3.1.

A detailed process diagram of pilot system 2 was shown in Figure 3.10. It operated without pre-ozonation. In addition, the upflow roughing filter (roughing filter A) and slow sand filter (slow sand filter 1) were designed according to recommendations in the literature (e.g. Wegelin, 1996), particularly with respect to bed depths and media size. However, the system also featured an additional parallel roughing filter (roughing filter B), which contained deeper bed depths of finer media for increased filtration efficiency, and an additional slow sand filter (slow sand filter 2) in series for increased robustness and protection against pathogen breakthrough. The concept of the additional slow sand filter was a contribution by Daniel Urfer, who tested a similar system in Switzerland. Both roughing filters operated in parallel to provide a comparison between the two designs. One of the roughing filters fed the downstream slow sand filters, while the other was ran to waste. A detailed description of pilot system 2 was given in Section 3.1.

5.2.2 Testing Methods

The two pilot systems were tested in parallel at the Mannheim Water Treatment Plant intake in the Regional Municipality of Waterloo, and fed with surface water from the Grand River, a

municipally and agriculturally impacted river in Southern Ontario. Pilot system 1 was commissioned in October 2003 and acclimatized for 4 months in cold water conditions, whereas pilot system 2 was commissioned in December 2003 and acclimatized for 2 months in cold water conditions before testing. During this start-up period, turbidity measurements were obtained with a portable handheld HACH 2100P turbidimeter.

After acclimatization, testing took place from February to May 2004, when a range of water temperatures (2 to 20^{o}C) and seasonally varying turbidity levels (0 to >100 NTU) occurred. Hydraulic loading rates in pilot system 1 (train 1), pilot system 1 (train 2), and pilot system 2 were varied from 0.2 to 0.4 m/h, 0.2 to 0.8 m/h, and 0.2 to 0.6 m/h, respectively. Such high hydraulic loading rates were tested to determine system performance under the potential scenario of taking one of two parallel units offline for cleaning, thus doubling the flow rate through the one system still in operation. Furthermore, in pilot system 1, train 1 operated with pre-ozonation from Feb. 25 to May 31, and train 2 operated with pre-ozonation from Mar. 22 to May 31.

Continuous turbidity data was collected at 15 minute intervals with HACH 1720D online turbidity meters, where each datum was the average of turbidity over the previous 15 minutes (calibration of the turbidity meter was discussed in Section 3.2.2). Online turbidity meters were located on the influent of pilot system 1, the effluent of both trains in pilot system 1, and the effluent of pilot system 2. The influent turbidity was assumed to be similar for both pilot systems, which were operating in parallel. Based on a two-tailed paired t-test (5% significance level) comparing handheld influent turbidity data form both pilots, this assumption was valid (see Appendix C: Table C.1). Thus, the online turbidity data from the pilot system 1 influent was applicable to both systems.

At various times throughout the study, the location of the effluent turbidity meters was switched between the effluent of the slow sand filter and the effluent of the following filter. In the case of pilot system 1, the filter following the slow sand filter was the GAC filter, whereas in the case of pilot system 2, it was a second slow sand filter, namely slow sand filter

2. This was done to determine the impact of the additional stages of filtration in the reduction of turbidity.

Headloss data was acquired by measuring the water levels above the filters in pilot system 1 and in various piezometers located throughout pilot system 2. In pilot system 1, the water level in the secondary ozone contactor and above the slow sand filter was monitored to assess the headloss across the roughing filter, and determine when cleaning was required (maximum water level reached). The datum of the water level measurements was the floor (bottom of the filters). Details of the piezometer locations in pilot system 2 were shown in Figure 3.10. The datum for the water level measurements in the roughing filters and slow sand filter 1 was the top of sand bed in slow sand filter 1. The datum for the water level measurements in slow sand filter 2 was the top of the sand bed in slow sand filter 2.

5.3 RESULTS AND DISCUSSION

5.3.1 Turbidity Data during the Start-up Period

Pilot system 1 was commissioned on October 7, 2003, and biologically acclimatized for 4 months before testing was initiated on February 9, 2004. It is likely that the system was not fully biologically mature prior to testing. Pilot system 2 was commissioned on December 19, 2003, and also likely still undergoing acclimatization when testing initiated on February 9, 2004. The handheld turbidity data during the start-up phase is shown on a logarithmic scale in Figure 5.1.

During the start-up phase, pilot system 1 operated continuously and endured various perturbations. Such perturbations included fluctuations in flow and applied ozone dose, pump failures, cleaning operations, and fluctuations in influent turbidity. Data during these events were not analyzed due to its highly variable nature and the number of influential factors involved.

During the first month of operation (Oct. 7 to Nov. 7) of pilot system 1, the raw water turbidity ranged from 2.5 to 30.6 NTU (average of 8.5 NTU) and the raw water temperature

ranged from 14 to 17.5°C. During this time, train 1 had effluent turbidities ranging from 0.22 to 5 NTU and averaged 0.87 NTU. Train 2 had effluent turbidities ranging from 0.21 to 2.37 NTU and averaged 0.65 NTU. The highest effluent turbidities of 5 NTU (train 1) and 2.37 (train 2) occurred on Oct. 17 during a rain event where the raw water turbidity was 14.2 NTU. As the filters had only been in operation for one week at this time, the schmutzdecke layer at the top of the sand bed was not yet well established, thus variations in effluent turbidity resulted from variations in raw water turbidity.

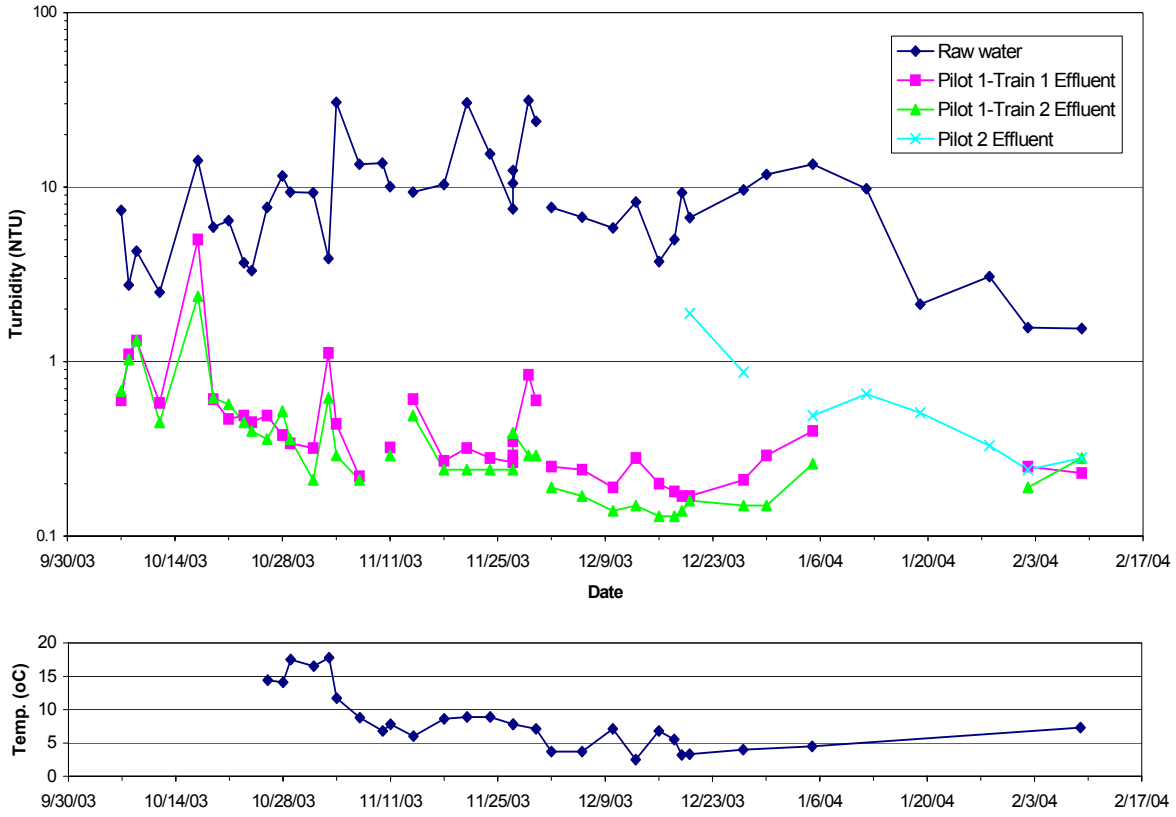

Figure 5.1: Turbidity Data during Start-up Period

Interestingly, three weeks later on Nov. 7, the raw water turbidity was again close to 14 NTU, however, in this case the effluent turbidities of train 1 and train 2 had reached the lowest of the month, 0.22 and 0.21 NTU, respectively. Clearly, the filters had acclimatized sufficiently to be able to produce more stable effluent turbidities during these fluctuations in raw water turbidity. However, effluent turbidities continued to fluctuate long after this event, and it was evident that the filters had not yet been fully acclimatized.

In Figure 5.1, a generally decreasing trend in effluent turbidity is evident in both train 1 and train 2, most likely due to the continued biological acclimatization of the filters, despite a decrease in influent water temperature. Effluent turbidities from train 1 and 2 reached a minimum around December 20, just over two months since start-up, with effluent values of 0.16 and 0.14 NTU, respectively. At this time, although the raw water temperatures had decreased to about 9^{o}C, it was assumed that the filters had been sufficiently acclimatized and were ready for extensive testing.

It was also observed that the effluent turbidity from train 2 was consistently slightly less than train 1, and less sensitive to raw water turbidity than train 1. This could have been due to slight differences in the media characteristics, although the same media was used in both trains.

Pilot system 2 was commissioned on Dec. 20, 2003 and a decreasing trend in effluent turbidity was observed until a minimum effluent of 0.24 NTU was reached in February. During this time, the influent turbidity also had a decreasing trend, thus it was unknown whether the decreasing effluent trend was due to the decrease in influent turbidity or the acclimatization process, or a combination of both. It is important to note that the water temperature during the acclimatization process was less than 5^{o}C, thus it is likely that the filters were not yet fully acclimatized at the beginning of the testing period in February.

5.3.2 Online Turbidity

This section analyzes and discusses the cumulative removal of turbidity throughout the two multistage filtration systems. The raw data used to generate the following graphs can be provided by the author on request, as the data set is too large for the Appendix. The results from the ensuing statistical analysis are shown in Appendix B (Table B.1 to B.3).

Runoff events, hydraulic loading rates through the slow sand filters, and the location of the online effluent turbidity meter are shown on the graphs. For pilot system 1, the location of the

effluent turbidity meter was the effluent of either the slow sand filter or the GAC filter. For pilot system 2, the location of the effluent turbidity meter was the effluent of either slow sand filter 1 or slow sand filter 2. In addition, the influent water temperatures are indicated below each graph.

Various events of operational disturbances are shown where applicable, such as filter cleaning events, sudden increases in hydraulic loading rate, GAC fines in the effluent, and periods of reduced flow rate due to clogging of the slow sand filter, which caused the headwater above the filter to overflow to waste.

Turbidity is shown on a logarithmic scale on all graphs. The current Ontario drinking water turbidity standard of 0.5 NTU for conventional treatment (MOE, 2003) is shown, as well as the future proposed Canadian turbidity guideline of 0.3 NTU (a new Canadian guideline would likely be adopted as a standard by Ontario). It is important to note, however, that the current Ontario effluent turbidity standard for slow sand filtration is 1 NTU (MOE, 2003).

Any periods of missing data were as a result of insufficient flow to the turbidity meter, thus resulting in unreliable turbidity data, which was not included in the analysis.

5.3.2.1 Pilot System 1 (Train 1 and Train 2): February 9 – April 9, 2004

Figure 5.2 shows the influent and effluent turbidity from both trains in pilot system 1 for the time period of February 9 to April 9. This phase of study presented a period of cold water conditions where the water temperature was below 5^{o}C for most of February and March, and rose to between 5 and 10^{o}C at the beginning of April. It is recalled that the filters were likely not fully biologically mature at the beginning of this study period.

During this phase of study, the hydraulic loading rate through the slow sand filter in each train was 0.4 m/h, the maximum recommended filtration rate for slow sand filtration according to the literature, with the exception of during major runoff events when it was reduced to 0.2 m/h. Two major runoff events occurred, the snowmelt and spring runoff beginning on March

2, which was immediately followed by a major rain event, and a series of rain events lasting one week from March 21 to March 31.

It is noted that pre-ozonation in train 1 commenced on Feb. 25, while pre-ozonation in train 2 commenced on Mar. 22. It is also noted that throughout the study, there were periods of time when the GAC filter was suspected to be producing turbidity through the escape of GAC fines in the filter effluent. In particular, from Apr. 4 to Apr. 8, the effluent data from train 1 was inexplicably high, reaching 10 NTU in some cases. Since there were no known operational disturbances during this time and influent turbidity was actually below 10 NTU, these data were treated as outliers and omitted from analysis.

Performance During Colder Water Temperatures

During colder water temperatures (<5^{o}C) from Feb. 9 to Mar. 1, with an average influent turbidity of 3.2 NTU, and slow sand filtration rate of 0.4 m/h, the effluent was below 0.5 NTU in 99.6% and 98.7% of the measurements for train 1 and train 2, respectively. Furthermore, it was below 0.3 NTU in 88.4% and 88.2% of measurements for train 1 and train 2, respectively. The average effluent turbidity of train 1 and 2 was 0.26 NTU and 0.28 NTU, with a standard deviation of 0.17 and 2.24 NTU, respectively. It is noted that the these results were generated from the overall data set collected from Feb. 9 to Mar. 1, and in the case of train 2, included effluent data from periods when the effluent from the slow sand filter was measured, and from a period when the effluent from the GAC filter was measured. It was not evident that the GAC filter provided additional removal of turbidity beyond that achieved by the slow sand filter, possibly because it had not been fully commissioned at this time (GAC fines still escaping the filter).

It will be recalled that this performance was measured after the filters had been in operation for only a few months during cold water temperatures. Despite exceptional performance, the filters were likely not fully biologically mature, and also susceptible to reduced biological treatment in cold water conditions.

Although the influent turbidity ranged from approximately 1.5 to 20 NTU, most of the variability in effluent turbidity data was believed to be associated with cleaning of the filters in both trains and the escape of GAC fines from the newly commissioned GAC filter in train 2. Cleaning of both trains occurred rather frequently in the beginning of the study (eg. cleaning on Feb. 14, 16, 20, and 21) as initial cleaning practices were not efficient in removing the schmutzdecke and subsequent headloss from the sand filters. Cleaning practices were subsequently revised and resulted in effective restoration of initial headloss values and increased filter run length.

The effect of GAC fines on effluent turbidity was evident from Feb. 23 to Feb. 29, when the turbidity meter was located on effluent of the GAC filter, instead of the slow sand filter effluent, on train 2 only. During this time, the effluent turbidity from train 2 had sudden and short-lived increases in turbidity to levels ranging from 5 to 100 NTU, while the effluent from train 1 remained below 0.5 NTU. Thus, the effluent turbidity from train 2 during this time should be disregarded and attributed to the commissioning of the post-GAC filter, and not to overall multistage filter performance.

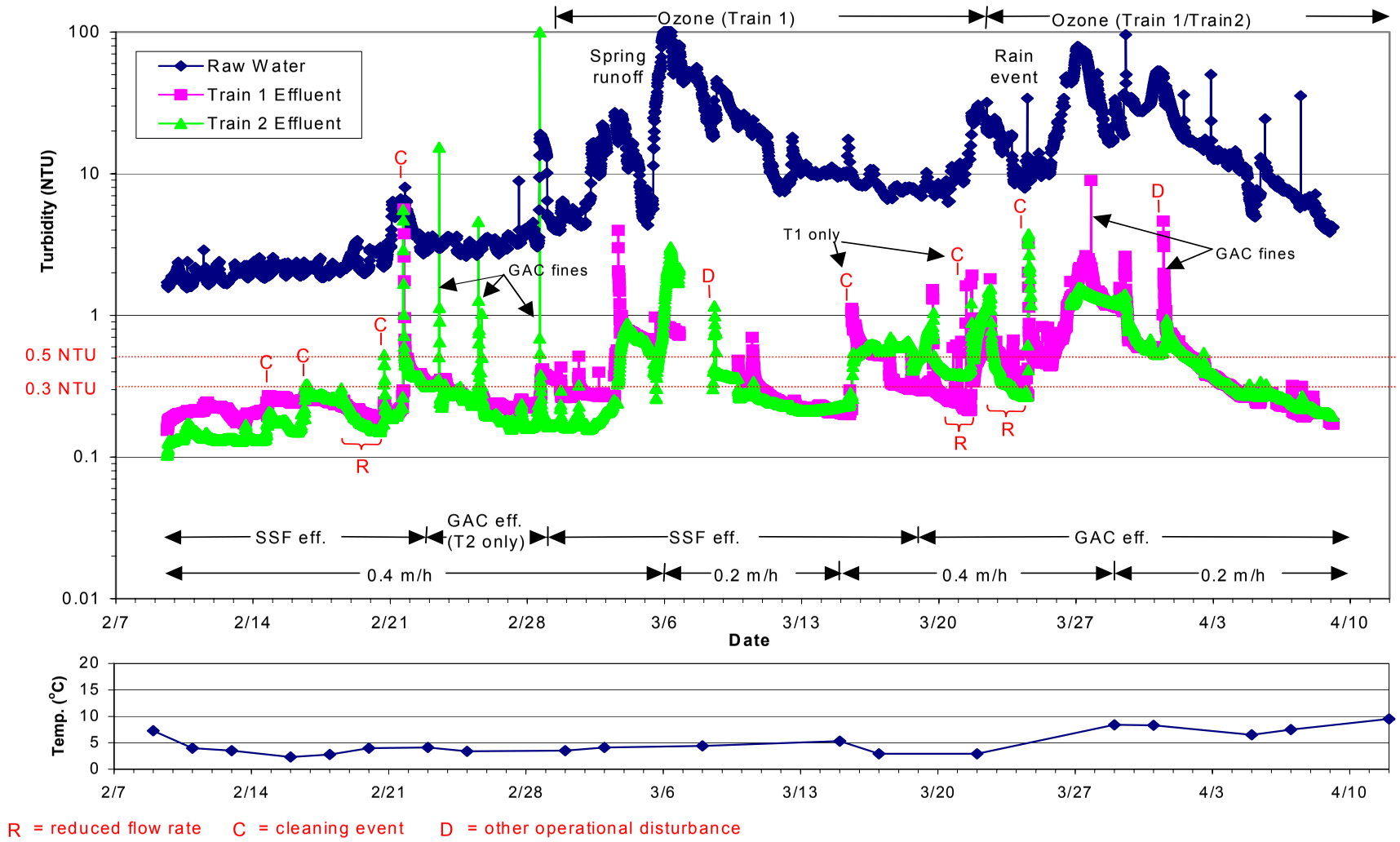

Figure 5.2: Pilot System 1 – Online Turbidity Data: February 9 – April 9, 2004

Another interesting period of cold water operation (<5^{o}C) took place from Mar. 12 to 21. During this time, the influent turbidity was rather consistent with an average of 9.4 NTU and a standard deviation of 1.9 NTU. From Mar. 12 to 15, the hydraulic loading rate through the slow sand filter was 0.2 m/h, resulting in an average effluent of 0.22 NTU and consistently below 0.3 NTU in both trains. However, on Mar. 15, the slow sand filter in train 1 was cleaned, after which the hydraulic loading rate was boosted to 0.4 m/h. This resulted in an immediate surge in effluent turbidity from 0.22 NTU and 0.27 NTU in train 1 and 2, respectively, to 1.12 NTU and 0.55 NTU. Clearly, the cleaning and the increased filtration rate had a compounded effect to cause an effluent turbidity greater than 1 NTU in train 1, while the sudden increase in filtration rate caused an increase to an effluent above 0.5 NTU in train 2. Thereafter, train 1 and train 2 operated with average effluent turbidities of 0.41 NTU and 0.54 NTU, respectively, compared to an average of 0.22 NTU before the increase in hydraulic loading rate. Thus, increasing the hydraulic loading rate to 0.4 m/h had a negative effect on effluent turbidity, although, while the influent turbidity remained below 10 NTU, the effluent turbidity in both trains was still consistently below 1 NTU.

Performance during Major Runoff Events

From Feb. 9 to Apr. 9, two major runoff events occurred: the snowmelt and spring runoff beginning on March 2, which was immediately followed by a rain event that lasted until Mar. 11, and a series of rain events lasting one week from March 21 to March 31. At some time during both events, the hydraulic loading rate through the slow sand filter was reduced from 0.4 m/h to 0.2 m/h to improve performance. Effluent turbidity performance and operational factors/disturbances during these major runoff events are discussed in the following section.

Spring Runoff:

On Mar. 2, a large snowmelt occurred and a spring runoff caused a surge in raw water turbidity to about 30 NTU on Mar. 3. The raw water turbidity then gradually decreased to about 5 NTU when only two days later, on Mar. 5, a large rain event occurred and raw water turbidity rapidly increased to over 100 NTU, beyond the maximum detection limit of the turbidity meter. The raw water turbidity soon decreased to below 100 NTU, although it remained above 20 NTU for several days and did not return to below 10 NTU until Mar. 11.

During this runoff event, the slow sand filter operated at a hydraulic loading rate of 0.4 m/h from Mar. 2 until Mar. 6, and thereafter operated at 0.2 m/h.

Overall, throughout the entire runoff event from Mar. 2 to Mar. 11, the average influent turbidity was 31 NTU with a standard deviation of 23 NTU. The effluent turbidity in train 1 and 2 averaged 0.53 NTU and 0.57 NTU, with a standard deviation of 0.31 and 0.55 NTU, respectively. Furthermore, the effluent turbidity was below 1 NTU in 98.5% and 90% of the measurements in train 1 and 2, respectively. However, before the hydraulic loading rate was reduced to 0.2 m/h on Mar. 6, the effluent turbidity in train 1 and 2 averaged 0.63 NTU and 0.76 NTU, with a standard deviation of 0.32 and 0.67 NTU, respectively. In this case, it was below 1 NTU in 97.7% and 82.6% of the measurements in train 1 and 2, respectively. This performance is still remarkable considering the high levels of influent turbidity, and the fact that the filters were likely not fully mature at this time.

It is evident from the graph in Figure 5.2 that the peaks in effluent turbidity in train 1 and 2 correlated well with the peaks in influent turbidity. However, both trains responded differently to sudden increases in influent turbidity. For example, on Mar. 3, with influent turbidity ranging from 15 to 25 NTU and a slow sand filtration rate of 0.4 m/h, train 1 effluent peaked at 3.98 NTU, however 10.5 hours later, train 2 reached a much lower peak of 0.88 NTU. Furthermore, on Mar. 6, with influent turbidity above 100 NTU, train 1 effluent peaked at 0.88 NTU, while train 2 effluent reached a much higher peak of 3.1 NTU. Clearly, the response of effluent turbidity to increased influent turbidity was inconsistent between both trains. It is also evident that spikes of increased effluent turbidity well above 1 NTU during these runoff events of high turbidity may partly be due to the relatively high hydraulic loading rate of 0.4 m/h through the slow sand filters. Furthermore, it is suspected that inadequate biological maturity of the filter and reduced biological treatment during cold water conditions had a negative impact on filter efficiency. This resulted in a high sensitivity of effluent turbidity to increased influent turbidity, thus allowing such high spikes in effluent turbidity. It is expected that with a fully mature filter, the effluent would be less sensitive to high levels of turbidity, especially during colder water temperatures. However, this remains to be determined.

Literature suggests that, during runoff events of high influent turbidity, effluent performance is improved when slow sand filters are operated at lower hydraulic loading rates. Thus, on Mar. 6, when the influent turbidity was approximately 75 NTU, the hydraulic loading rate was reduced to 0.2 m/h in both trains. Unfortunately, after the flow rate was reduced, the amount of flow that could be diverted from the main flow for proper operation of the turbidity meter was insufficient, thus effluent turbidity data during this time was unreliable and omitted. Therefore, the effect of a sudden reduction in the hydraulic loading rate could not be determined. Reliable measurements of effluent turbidity resumed on Mar. 8 in train 2 and Mar. 9 in train 1 when all of the effluent flow was directed through the turbidity meter. At this time, the effluent turbidity had decreased to below 0.5 NTU, except for a short-lived spike to 1.2 NTU in train 2, however the influent turbidity had also decreased to about 20 NTU, thus the decrease in effluent turbidity could not be attributed entirely to the decrease in hydraulic loading rate.

It is important to note the reasoning for the short-lived effluent spike of 1.2 NTU in train 2 on Mar. 8. This was caused by a sudden increase in hydraulic loading rate through the slow sand filter, which resulted from sampling an excessive flow of water from the bottom of the filter. Within 4 hours from the time of sampling, the effluent turbidity had returned to less than 0.5 NTU. Again, had the filters been more biologically mature, it is suspected that the resulting spike would have been lower, and perhaps below 1 NTU.

Series of Rain Events:

During the series of rain events that lasted from Mar. 21 to Apr. 4, the raw water turbidity peaked at 32 NTU on Mar. 22, 79 NTU on Mar. 27, 96 NTU on Mar. 29, and 53 NTU on Mar. 31. The slow sand filters operated at a hydraulic loading rate of 0.4 m/h from Mar. 21 to Mar. 29, and thereafter operated at 0.2 m/h. The effluent turbidity meter was located after the GAC filter during this time, and it was unclear whether the GAC filter improved the effluent turbidity from the slow sand filter. In fact, there were some occurrences of high effluent turbidity where it was suspected that the GAC filter was producing turbidity, likely due to the escape of GAC fines from the filter.

Overall, the average influent turbidity from Mar. 21 to Apr. 4 was 25 NTU with a standard deviation of 16 NTU. The effluent turbidity in train 1 and 2 averaged 0.73 NTU and 0.72 NTU, with a standard deviation of 0.56 and 0.44 NTU, respectively. Furthermore, it was below 1 NTU in only 78% and 72% of the measurements in train 1 and 2, respectively.

Clearly, the performance of the multistage filter was not as good during this major runoff event, compared to the spring runoff event. However, there are a number of operational factors or disturbances that could have contributed to poor effluent turbidity, beyond that of just increased influent turbidity. For example, on Mar. 21, the slow sand filter in train 1 was cleaned, resulting in a spike of effluent turbidity to 1.9 NTU. Similarly, on Mar. 24, the slow sand filters in both trains were cleaned and disturbance of the GAC filter presumably caused GAC fines to enter the turbidity meter. As a result, both trains experienced a spike in effluent turbidity to approximately 3 NTU. In addition, on Mar. 31, due to the observation of a high amount of GAC fines in the effluent of train 1, the GAC filter was drained down and refilled by the operator. However, when the GAC filter was drained, a large head differential between the water level in the slow sand filter and that of the GAC filter created a surge in hydraulic loading rate through the slow sand filter. This resulted in a spike of effluent turbidity reaching 4.6 NTU.

Due to continued poor filter performance from Mar. 26 to 29, with effluent turbidities consistently above 1 NTU in both trains, the hydraulic loading rate through the slow sand filter was reduced from 0.4 m/h to 0.2 m/h on Mar. 29. Although the influent turbidity remained around 30 NTU, the effluent turbidity in both trains immediately decreased from 1 NTU to less than 0.6 NTU, despite a subsequent increase in influent turbidity to approximately 50 NTU. This suggests that, during runoff events of high turbidity, lowering the hydraulic loading rate to typical literature recommended values (i.e. 0.2 m/h) was important for achieving an effluent turbidity below 1 NTU.

The Effect of Pre-Ozonation on Effluent Turbidity

Pre-ozonation in train 1 and 2 commenced on Feb. 25 and Mar. 22, respectively, thus a period of time from Feb. 25 to Mar. 22 occurred, where only train 1 was receiving pre-ozonation, thus the effect of pre-ozonation on effluent turbidity could be isolated and determined. During this time, the average effluent turbidity of train 1 and 2 was 0.38 and 0.47 NTU (standard deviation of 0.23 and 2.12 NTU), respectively. Based on a paired t-test with a 5% significance (2162 data points), the difference between train 1 and train 2 was significant, however only by a minimal margin. Thus, the paired t-test was performed again, but this time with a 1% level of significance, and the difference between train 1 and train 2 was indeed not significant (see Appendix B: Table B.2). Thus, it was determined that pre-ozonation had no significant effect on effluent turbidity

Summary of Results from Pilot System 1 (February 9 – April 9)

During the colder water temperatures (<5 to 10^{o}C) from Feb. 9 to Apr. 9, with an average influent turbidity of 14 NTU and standard deviation of 18 NTU, effluent measurements were below 1 NTU in 92.7% and 90.1% of the measurements for train 1 and train 2, respectively. It was below 0.5 NTU in 86.5% and 84.5% of the measurements, and below 0.3 NTU in 76.3% and 75.2% of the measurements for train 1 and 2, respectively. Furthermore, the average effluent turbidity of train 1 and 2 was 0.43 NTU and 0.45 NTU, with a standard deviation of 0.41 and 1.6 NTU, respectively. It is noted that these results included effluent data from periods when the effluent from the slow sand filter was measured, and from a period when the effluent from the GAC filter was measured. However, it was not evident that the GAC filter provided additional removal of turbidity beyond that achieved by the slow sand filter. Further, it is important to note that pre-ozonation had no statistically significant effect on effluent turbidity.

This performance was achieved despite a period of low water temperature (<5^{o}C), a number of runoff events (including spring runoff) with influent turbidity as high as 100 NTU, and periods of hydraulic loading as high as 0.4 m/h. In addition, at the beginning of this phase of study, the filters had been in operation for only a few months during cold water temperatures.

Thus, it is likely that the filters were not fully biologically mature, and also susceptible to reduced biological treatment in cold water conditions.

Performance during cold water temperatures (<5^{o}C) was excellent with raw water turbidities below 10 NTU. However, during runoff events with high influent turbidity and cold water temperatures, performance was not as good, as effluent turbidity increased above 1 NTU on a number of occasions. In addition, operational disturbances such as cleaning of the slow sand filter and sudden, short-lived increases in hydraulic loading rate caused temporary spikes of effluent above 1 NTU. The high sensitivity of effluent turbidity to increased influent turbidity and operational disturbances was likely due to inadequate biological maturity of the filter, decreased biological treatment due to cold water temperatures, and the relatively high hydraulic loading rate of 0.4 m/h. Under similar operating conditions, it remains to be seen whether improved performance could be achieved with a fully biologically mature filter.

Interestingly, during both major runoff events, decreasing the hydraulic loading rate to 0.2 m/h effectively reduced effluent turbidity to less than 1 NTU. Thus, lowering the hydraulic loading rate during runoff events of high turbidity combined with cold water temperatures was important for meeting the current Ontario standard for slow sand filter effluent. It remains to be determined whether this would be required with a fully mature filter.

5.3.2.2 Pilot System 1 (Train 1 and Train 2): April 9 – June 1, 2004

Figure 5.3 shows the influent turbidity and effluent turbidity from both train 1 and train 2 in pilot system 1 for the time period of April 9 to June 1. The water temperatures during this phase of study ranged between 5 and 20^{o}C. Three major rain events occurred, beginning on Apr. 18, May 10, and May 22, where raw water turbidities reached 55, 23, and 75 NTU, respectively. It is noted that pre-ozonation was present in both trains throughout this phase of study.

After the hydraulic loading rate was boosted from 0.2 m/h to 0.4 m/h on April 9, the slow sand filter in train 1 continued to operate at 0.4 m/h for the rest of the study period. However, train 2 was slowly ramped up from 0.4 m/h to 0.6 m/h, and finally to 0.8 m/h, where a

comparison of train 1 (0.4 m/h) and train 2 (0.8 m/h) took place for approximately 3 weeks. The ramp up period lasted just over 3 weeks to allow time for the biomass in the roughing filter and slow sand filter to acclimatize to the increased solids and organic loading conditions.

It is noted that the effluent online turbidity meter was located after the GAC filter in both trains from Apr. 9 to 23, and thereafter it was located after the slow sand filter. It was believed that the GAC filters were fully commissioned by Apr. 9, and no GAC fines were suspected in producing any unexpected and increased effluent turbidity.

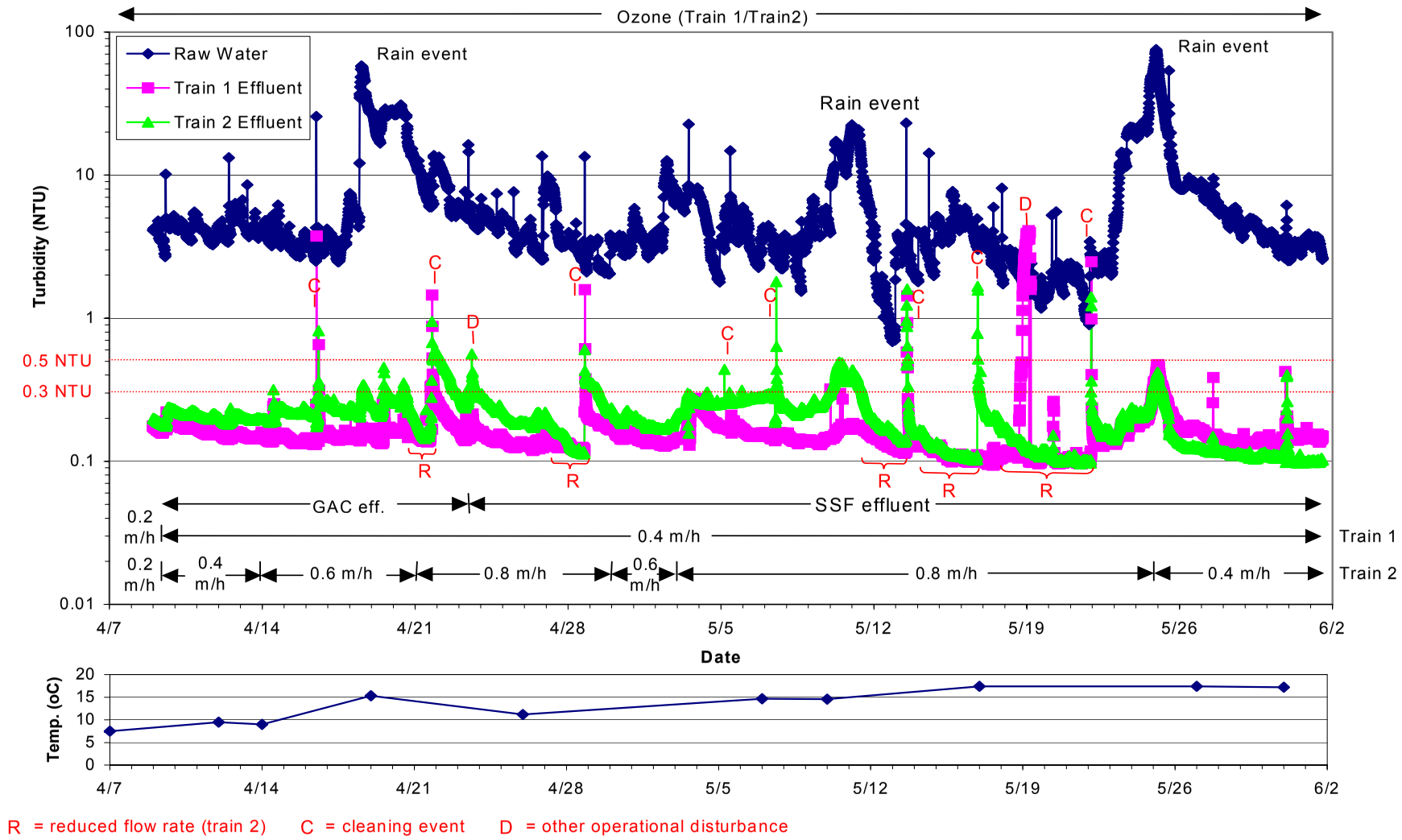

Figure 5.3: Pilot System 1 – Online Turbidity Data: April 9 – June 1, 2004

Performance During Warmer Water Temperatures, Major Runoff Events, and Increased Hydraulic Loading

During warmer water conditions (7.5 to 17.5^{o}C) from Apr. 9 to June 1, with an average influent turbidity of 6.7 NTU and standard deviation of 8.2 NTU, effluent measurements were below 0.5 NTU in 99.1% and 98.9% of the measurements for train 1 and train 2, respectively. Furthermore, it was below 0.3 NTU in 97.7% and 90.6% of measurements for train 1 and train 2, respectively. The average effluent turbidity of train 1 and 2 was 0.17 NTU and 0.21 NTU, with a standard deviation of 0.25 and 0.1 NTU, respectively. This analysis of effluent turbidity includes a period of time when the GAC filter effluent was measured and a period when slow sand filter effluent was measured. These results were pooled into the same analyzed data set, as it was not evident that the GAC filter had an impact on effluent turbidity.

It is important to note that there were a number of unexpected instances of reduced hydraulic loading through the slow sand filter in train 2, due to clogging of the sand filter and subsequent overflow of headwater to waste. This resulted in reduced effluent turbidities that were not representative of the pre-determined hydraulic loading conditions that were to be tested. Thus, a revised analysis of the effluent data from train 2 was done, in which data from periods of reduced hydraulic loading (from clogging of the sand filter) were omitted (22.5% of the total data set was omitted). The results were very similar to the initial results of train 2, including an effluent below 0.5 NTU in 98.5% of the measurements, and below 0.3 NTU in 87.9% of the measurements. In addition, the revised average effluent turbidity of train 2 was 0.23 NTU with a standard deviation of 0.1 NTU. These results are exceptional, especially considering that train 2 operated for over 3 weeks with a hydraulic loading as high as 0.8 m/h.

It is recalled that this performance was measured after the filters had been in operation for several months, thus it is likely that they were more biologically mature than during the months of February and March. Furthermore, the influent water temperatures were higher, thus increased biological treatment likely led to lower effluent turbidities. These results also included effluent data from a number of cleaning events, including a high frequency of cleaning in train 2 during its operation at 0.8 m/h, and operational disturbances, such as a

human induced effluent turbidity spike of 0.6 NTU on April 23 and a 4 NTU turbidity spike of unknown origin on May 19.

More importantly, during three major rain events with influent turbidities ranging from 20 to 75 NTU, the multistage filter effluent from both trains remained below 0.5 NTU, even with hydraulic loading rates ranging from 0.4 to 0.8 m/h. Performance during these rain events was far superior to that during the major runoff events in February and March (as discussed in the previous section). In addition, reducing the hydraulic loading rate to 0.2 m/h during these events was not necessary for achieving effluent turbidity below 1 NTU, as it was during the runoff events in February and March. This suggests that increased biological maturity of the filters, as well as increased water temperature and subsequent biological activity, were likely important factors in multistage filter performance during events of high influent turbidity.

The Effect of Increased Hydraulic Loading Rate during Warm Water Conditions

The following section analyzes the impact of increased hydraulic loading rate on effluent turbidity during periods of warmer water temperatures. Data from Apr. 21 to 29 and May 3 to 25 were analyzed to determine if any difference existed between the effluent turbidity from train 1, which had a hydraulic loading of 0.4 m/h through the slow sand filter, compared to train 2, which had a hydraulic loading 0.8 m/h. During this time, all other operating conditions were equal between both trains.

The average effluent turbidity from train 1 and train 2 was 0.19 NTU and 0.28 NTU, with a standard deviation of 0.32 and 0.12 NTU, respectively. The results from both trains were excellent during this time period, especially for train 2, considering that it operated at a hydraulic loading rate of 0.8 m/h. However, with a paired t-test using a 1% level of significance, the small difference in effluent turbidity between train 1 and 2 was significant, suggesting that the increased hydraulic loading rate had a negative impact on effluent turbidity. Nevertheless, 96.9% of the effluent measurements in train 2 were still below 0.5 NTU, similar to 98.5% in train 1. Again, it is suspected that increased biological maturity of the filter, as well as increased water temperatures and subsequent biological treatment likely had an important role in maintaining effluent turbidities below 0.5 NTU. However, it

remains to be determined whether similar performance at a hydraulic loading rate of 0.8 m/h can be achieved in a fully biologically mature filter at lower water temperatures (<5^{o}C).

Summary of Results from Pilot System 1 (April 9 – June 1)

During warmer water conditions (7.5 to 17.5^{o}C) from Apr. 9 to June 1, with an average influent turbidity of 6.7 NTU and standard deviation of 8.2 NTU, the multistage filter effluent was below 0.5 NTU in 99.1% and 98.5% of the measurements for train 1 and train 2, respectively. Furthermore, it was below 0.3 NTU in 97.7% and 87.9% of measurements for train 1 and train 2, respectively. The average effluent turbidity of train 1 and 2 was 0.17 NTU and 0.23 NTU, with a standard deviation of 0.25 and 0.1 NTU, respectively. This analysis of effluent turbidity includes a period of time when the GAC filter effluent was measured and a period when slow sand filter effluent was measured. These results were pooled into the same analyzed data set, as it was not evident that the GAC filter had an impact on effluent turbidity.

This performance was achieved despite experiencing three major runoff events with influent turbidities ranging from 20 to 75 NTU, and hydraulic loading rates ranging from 0.4 m/h (train 1) to 0.8 m/h (train 2). During these challenging conditions, the multistage filter effluent from both trains remained below 0.5 NTU. This was likely attributable to increased biological maturity of the filter, and increased water temperatures and subsequent biological treatment, resulting in lower sensitivity of effluent turbidity to large fluctuations in influent turbidity. It remains to be determined whether similar performance at a hydraulic loading rate of 0.8 m/h can be achieved in a fully biologically mature filter at lower water temperatures (<5^{o}C).

Finally, under warmer conditions and a more mature filter, reducing the hydraulic loading rate to 0.2 m/h during these runoff events was not necessary for achieving effluent turbidity within regulation.

5.3.2.3 Pilot System 2: February 27 – March 15, 2004

Figure 5.4 shows the influent turbidity and effluent turbidity from pilot system 2 for the time period of February 27 to March 15. This phase of study presented a period of cold water conditions where the water temperature was below 5^{o}C for most of February and the beginning of March, and only slightly above that value later in March. It is noted that there was no pre-ozonation in pilot system 2, and the roughing filter and slow sand filter 1 featured deeper bed depths than in pilot system 1.

The hydraulic loading rate through the slow sand filter was 0.4 m/h, the maximum recommended filtration rate for slow sand filtration according to the literature, until Mar. 9, when it was reduced to 0.2 m/h and continued to operate at this hydraulic loading until Mar. 15. One major runoff event occurred during this phase of the study, which involved the snowmelt and spring runoff beginning on March 2 that was immediately followed by a rain event causing high raw water turbidity for approximately one week.

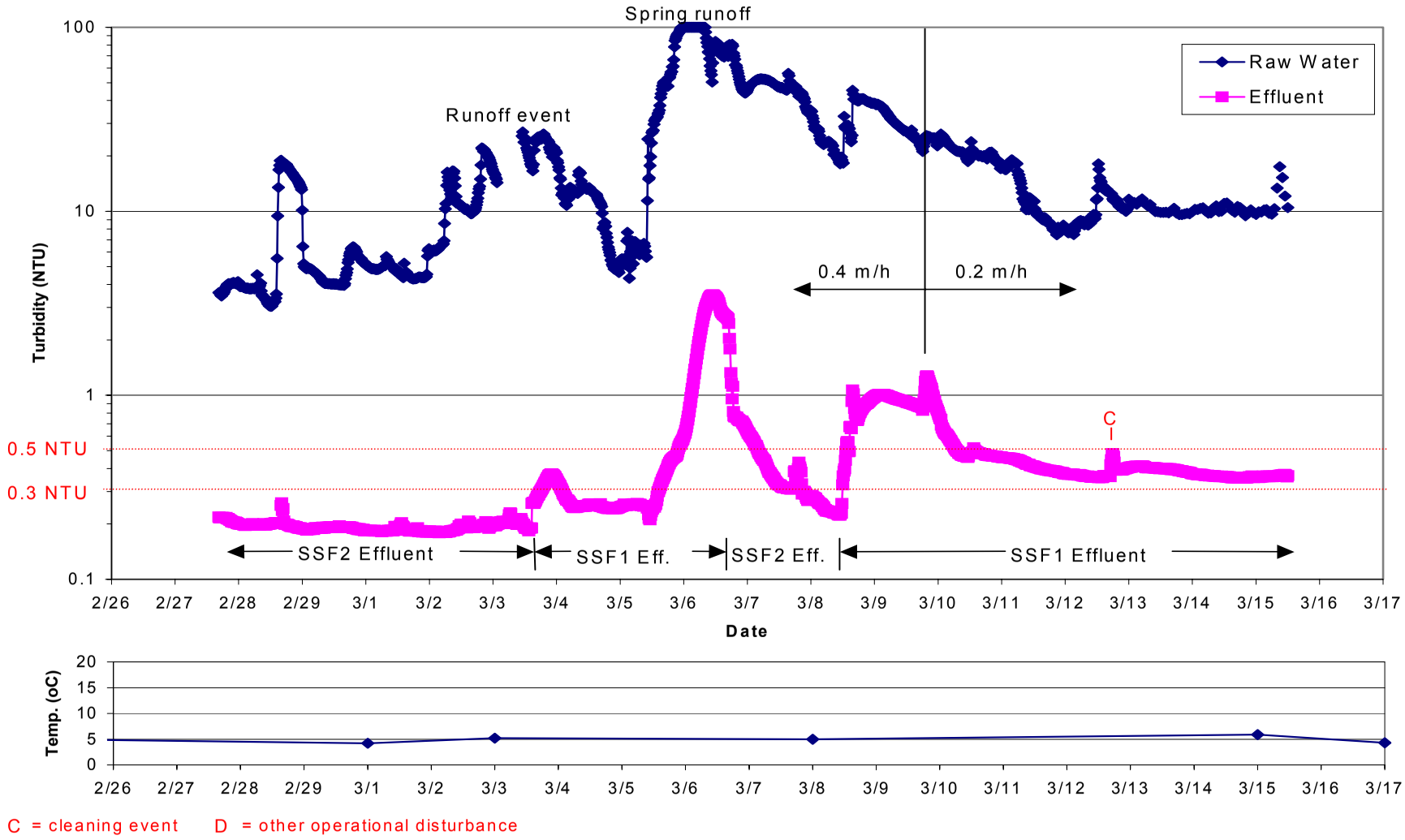

Figure 5.4: Pilot System 2 – Online Turbidity Data: February 27 – March 15, 2004

Performance During Colder Water Temperatures

During cold water temperatures (<5^{o}C) from Feb. 27 to Mar. 15, with an average influent turbidity of 22 NTU and standard deviation of 22 NTU, and hydraulic loadings ranging from 0.2 to 0.4 m/h, the multistage filter effluent was below 1 NTU in 94.3% of the measurements, below 0.5 NTU in 82.2% of the measurements, and below 0.3 NTU in 42.7% of the measurements. The average effluent turbidity was 0.46 NTU, with a standard deviation of 0.5 NTU. It is recalled that this performance was measured after the filters had been in operation for only a two months during cold water temperatures. Thus, the filters were likely not fully biologically mature, and also susceptible to reduced biological treatment in cold water conditions.

It is important to note that this analysis is conservative as it includes pooled data from periods when the turbidity meter was located after slow sand filter 1, and periods when it was located after slow sand filter 2. In addition, it includes data from operation at both 0.4 m/h and 0.2 m/h. Thus, rather than drawing conclusions from the pooled data set, the following section further analyzes the results from isolated periods of specific operational conditions.

From Feb. 27 to Mar. 3, while the turbidity meter was located after slow sand filter 2, the effluent from slow sand filter 2 was consistently below 0.3 NTU, even with periods of influent turbidity as high as 27 NTU, and temperatures below 5^{o}C. On Mar. 3, however, the location of the turbidity meter was switched to the effluent of slow sand filter 1 and there was an immediate increase in effluent measurements from 0.19 to 0.38 NTU, however the slow sand filter 1 effluent was still below 0.5 NTU, even with influent turbidities of approximately 25 NTU, and soon after dropped back down to continuously produce effluent below 0.3 NTU until the runoff event on starting on Mar. 5.

During the major spring runoff event starting on Mar. 5, raw water turbidities reached over 100 NTU on Mar. 6, beyond the maximum detection limit of the online turbidity meter. This caused a spike in the slow sand filter 1 effluent to 3.5 NTU. Just after the peak in effluent turbidity was beginning to subside, the location of the effluent turbidity meter was switched back to the effluent of slow sand filter 2. This resulted in an immediate and dramatic

reduction in effluent measurements from 2.5 to 0.75 NTU, less than the Ontario standard of 1 NTU for slow sand filters. Thus, the second slow sand filter in series was important for achieving effluent turbidity below 1 NTU during this event of high raw water turbidity and a relatively high hydraulic loading of 0.4 m/h.

From 8:00 pm on Mar. 6 to 11:30 am on Mar. 8, the slow sand filter 2 effluent continued to decrease below 0.5 NTU, despite influent turbidities as high as 56 NTU, and further decreased to below 0.3 NTU as the influent turbidity decreased to approximately 20 NTU. These effluent measurements from slow sand filter 2 were remarkable considering the high influent turbidity, relatively high hydraulic loading, and the inadequate biological maturity of the system.

On Mar. 8, however, the location of the effluent turbidity meter was switched back to the effluent of slow sand filter 1, and there was an immediate increase in effluent measurements to above 1 NTU. However, there was also a sudden increase in raw water turbidity to 45 NTU. Thus, the dramatic increase in effluent turbidity was likely due to a combination of these two factors. It is important to recall however, that slow sand filter 2 achieved an effluent turbidity of less than 0.5 NTU just one day earlier, with an even higher influent turbidity of 56 NTU.

More importantly, on Mar. 9, while the effluent of slow sand filter 1 was still above 1 NTU, the hydraulic loading rate was reduced from 0.4 to 0.2 m/h and the effluent immediately returned to below 1 NTU. Thus, during this event of increased influent turbidity, reducing the hydraulic loading rate to 0.2 m/h was important for producing a slow sand filter 1 effluent below 1 NTU.

For the rest of this phase of study, the effluent turbidity from slow sand filter 1 continued to decrease to below 0.5 NTU as the raw water influent decreased to approximately 10 NTU. Even when the slow sand filter was cleaned (by scraping the schmutzdecke) on Mar. 12, the resulting effluent spike was still below 0.5 NTU, likely due to the relatively lower influent turbidity and lower hydraulic loading rate of 0.2 m/h.

Summary of Results from Pilot System 2 (February 27 – March 15)

During cold water temperatures (<5^{o}C) from Feb. 27 to Mar. 15, with an average influent turbidity of 22 NTU and standard deviation of 22 NTU, and hydraulic loadings ranging from 0.2 to 0.4 m/h, the effluent was below 1 NTU in 94.3% of the measurements, and below 0.5 NTU in 82.2% of the measurements. The average effluent turbidity was 0.46 NTU, with a standard deviation of 0.5 NTU. It is important to note, however, that this analysis is conservative as it includes pooled data from periods when the turbidity meter was located after slow sand filter 1, and periods when it was located after slow sand filter 2.

Nevertheless, this performance was achieved despite experiencing a runoff event with influent turbidity as high as 100 NTU, and hydraulic loading rates ranging from 0.2 to 0.4 m/h. It will be recalled that this performance was measured after the filters had been in operation for only two months during cold water temperatures. Thus, the filters were likely not fully biologically mature, and also susceptible to reduced biological treatment in cold water conditions.

Although, without the second slow sand filter, pilot system 2 performed well at lower influent turbidities and water temperatures below 5^{o}C, it failed to produce effluent below 1 NTU during runoff events of high turbidity when operated at a hydraulic loading rate of 0.4 /h. This was likely due to inadequate biological maturation of the filter, decreased biological treatment at lower water temperatures, and the relatively high hydraulic loading rate. It remains to be seen whether a biologically mature pilot system 2, under similar operational conditions and in the absence of a second slow sand filter in series, could achieve effluent below 1 NTU.

The second slow sand filter in series proved to be important for providing additional robustness to the process by achieving effluent turbidity below 1 NTU during periods of high raw water turbidity, low water temperatures (<5^{o}C), and a relatively high hydraulic loading rate of 0.4 m/h. Alternatively, in the absence of the second slow sand filter, reducing the

hydraulic loading rate to 0.2 m/h was another method of achieving an effluent below 1 NTU during these challenging conditions.

5.3.2.4 Pilot System 2: March 14 – April 12, 2004

Figure 5.5 shows the influent turbidity and effluent turbidity from pilot system 2 for the time period from March 14 to April 12. This phase of study presented a period of cold water conditions where the water temperature was between 5^{o}C and 10^{o}C.

This phase of study featured a series of three runoff events of high turbidity in which the slow sand filter operated at a hydraulic loading rate of 0.6 m/h. Although the hydraulic loading rate was gradually ramped up from 0.2 m/h, it was only operating at 0.4 m/h for 4 days before it was increased to 0.6 m/h, and only 4 days at 0.6 m/h before the runoff events initiated. Thus, it is important to consider that it was unlikely that the biomass in the filters was fully acclimatized to the new solids and organic loading conditions before the runoff events initiated.

As an aside, it is noted that there were a number of occurrences throughout this phase of study when the raw water turbidity meter measured short-lived (15 minute), isolated, and unexplained spikes of relatively high turbidity that were not within the apparent trend of the data. These were suspected anomalies in the data set. Nevertheless, the effluent turbidity did not seem to be affected by these short-lived spikes.

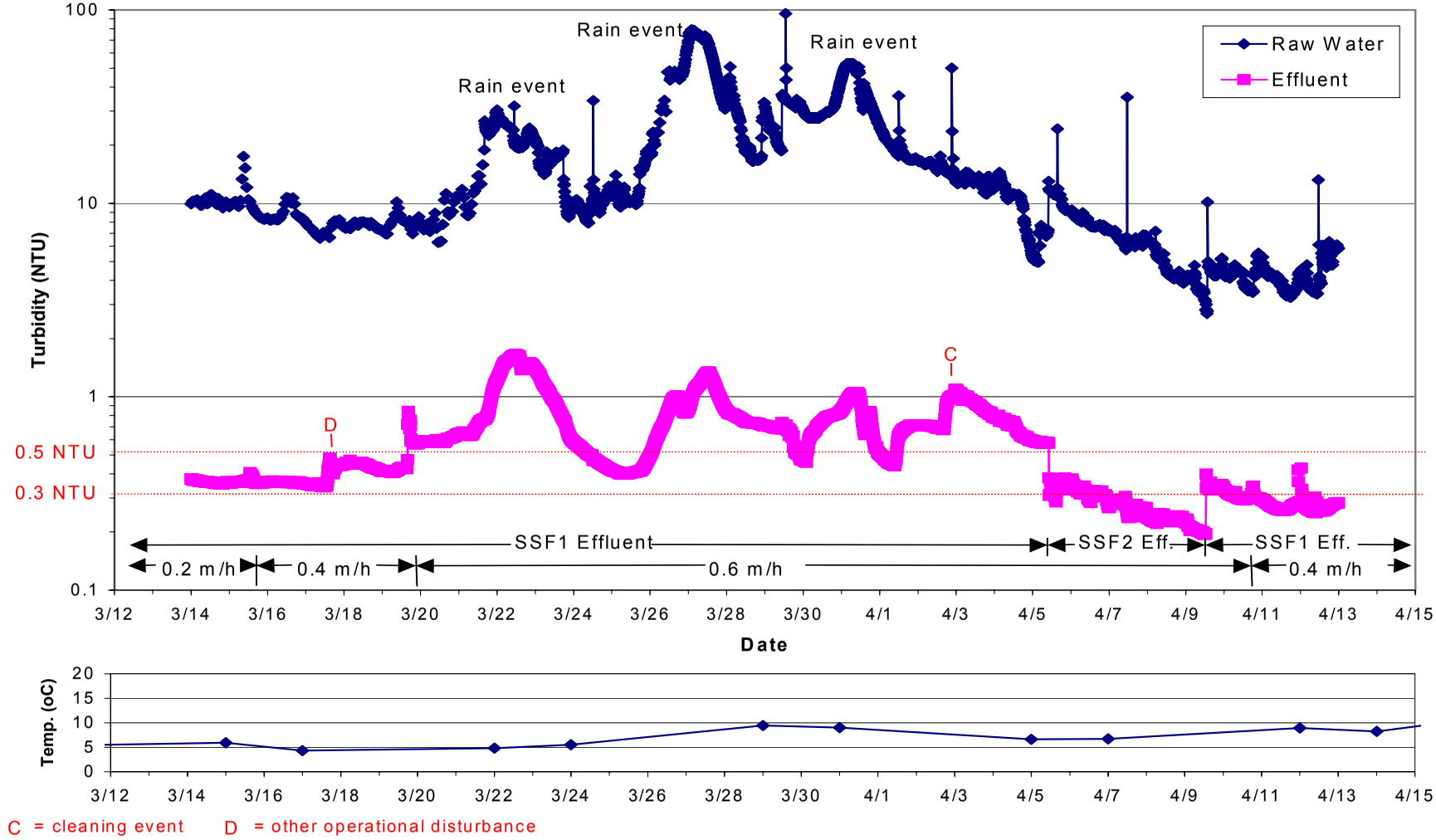

Figure 5.5: Pilot System 2 – Online Turbidity Data: March 14 – April 12, 2004

Performance during Runoff Events and Increased Hydraulic Loadings

On March 15, increasing the hydraulic loading from 0.2 to 0.4 m/h had little effect on the effluent turbidity from slow sand filter 1, as it remained below 0.5 NTU. Even a sudden and short-lived increase in hydraulic loading on Mar. 17 did not produce effluent above 0.5 NTU. This stability in effluent turbidity is likely due to the relatively low influent turbidity, which was approximately 10 NTU or less. However, when the hydraulic loading rate was further increased to 0.6 m/h only 4 days later, the effluent from slow sand filter 1 immediately increased to 0.84 NTU and thereafter stabilized at approximately 0.6 NTU. This level of effluent turbidity was sustained only until raw water turbidities increased to above 10 NTU at the onset of the runoff event on Mar. 21.

On Mar. 22, during the runoff event with an influent turbidity of approximately 30 NTU, the effluent from slow sand filter 1 increased dramatically and reached a peak of 1.66 NTU. The effluent exceeded 1 NTU likely partly due to the high hydraulic loading rate, but more importantly, because the biomass in the filter was not fully acclimatized to the new solids and organics loading conditions before the runoff event occurred.

Interestingly, 5 days later on Mar. 27, the raw water reached a much higher turbidity of approximately 80 NTU, yet this time, the effluent of slow sand filter 1 was somewhat lower, at 1.34 NTU. At that time, the filter had been operating at 0.6 m/h for one week, thus it was likely the filter was more biologically acclimatized to the increased loading conditions, thus resulting in a relatively lower effluent turbidity.

Nevertheless, even during the third runoff event with an influent turbidity of 52 NTU, occurring on Mar. 31, 12 days after operation at 0.6 m/h was commenced, the effluent from slow sand filter 1 was still above 1 NTU, at 1.05 NTU. Thus, during this event of high influent turbidity combined with an increased hydraulic loading of 0.6 m/h, a second slow sand filter in series would have likely been important for producing an effluent below 1 NTU. Unfortunately, the effluent from slow sand filter 2 was not measured during this time because of instrument limitations.

In addition, at such a high hydraulic loading rate, post-treatment with a second slow sand filter would have also been important during cleaning events, such as on April 2, when the schmutzdecke of slow sand filter 1 was scraped and removed, resulting in an effluent spike greater than 1 NTU. Practically speaking, it would have been prudent to lower the hydraulic loading rate after cleaning until the biofilm in the top of the sand filter had fully re-established.

In response to these postulations, the location of the effluent turbidity meter was switched to the effluent of slow sand filter 2 on Apr. 5, which produced a sudden decrease in effluent measurements from 0.57 to 0.38 NTU, below the current Ontario standard for chemically assisted filtration. This suggests that the second slow sand filter in series was important for providing additional removal of turbidity at the higher hydraulic loading rate of 0.6 m/h. At this hydraulic loading, it is likely that the impact of the second slow sand filter would be more profound during runoff periods of higher influent turbidity.

Summary of Results from Pilot System 2 (March 14 – April 12)

With water temperatures ranging from 5 to 10^{o}C, a hydraulic loading rate of 0.6 m/h, and average influent turbidity of 23.5 NTU (standard deviation of 15.6 NTU), the effluent from slow sand filter 1 was below 1 NTU in 82.2% of the measurements, and below 0.5 NTU in only 13.1% of the measurements. The average effluent turbidity was 0.78 NTU with a standard deviation of 0.27 NTU. This performance was achieved despite experiencing three runoff events with influent turbidities ranging from 30 to 80 NTU, and it was likely that the filters were not fully mature prior to this phase of study.

However, during each runoff event of high influent turbidity and a subsequent cleaning event, the effluent from slow sand filter 1 increased to above 1 NTU, the current regulatory standard for slow sand filtration in Ontario. Thus, operating the filter at a hydraulic loading rate of 0.6 m/h resulted in poor performance during these challenging events. Furthermore, at this hydraulic loading rate, the data suggests that pilot system 2, in the absence of the second sand filter in series, could only consistently produce an effluent below 1 NTU when raw water turbidity was less than 14 NTU.

However, it is suspected that performance would have been improved had the filters been fully acclimatized to the increased loading conditions. In fact, performance did improve with time, which suggests that filter acclimatization to the new loading conditions had a positive effect on effluent turbidity. However, in a full-scale application, a sudden increase in hydraulic loading is more likely to occur, rather than a gradual ramp up. For example, this would occur if one of two parallel systems were taken offline for cleaning, thus doubling the flowrate through the other system.

Finally, at the higher hydraulic loading rate of 0.6 m/h, the second slow sand filter was important for providing additional removal of turbidity beyond that achieved by slow sand filter 1. At this hydraulic loading, it is likely that the impact of the second slow sand filter in series would have been more profound during runoff periods of higher influent turbidity, however this remains to be determined.

5.3.2.5 Pilot System 2: April 13 – June 1, 2004

Figure 5.6 shows the influent turbidity and effluent turbidity from pilot system 2 for the time period of April 13 to June 1. This phase of study presented a period of warmer water conditions where the water temperature was generally above 10^{o}C and increased to a maximum of 17.5^{o}C by the end of May. Also, it is likely that the system was more mature at this time, as it was in operation for 5 months prior to this phase of study.

This section evaluates the performance of pilot system 2 in warm water conditions, especially during three runoff events with raw water turbidities ranging from 20 to 75 NTU. For the duration of this phase, the effluent turbidity meter was located on the effluent of slow sand filter 1, and the slow sand filter operated at a hydraulic loading rate of 0.4 m/h.

It is important to note that there were a number of occurrences when the raw water turbidity meter measured short-lived (15 minute), isolated, and unexpected spikes of relatively high turbidity that were not within the apparent trend of the data. These were suspected anomalies

in the data set. Nevertheless, the effluent turbidity did not seem to be affected by such short-lived spikes.

Also, during a period of time from May 14 to 17, the slow sand filter was overflowing to waste due to clogging, thus a reduced flow rate through the slow sand filter resulted in reduced effluent turbidities that were not representative of the pre-determined hydraulic loading condition that was to be tested. Thus, data from this time period was not included in the analysis.

Finally, from May 26 to 30, there was insufficient flow to the effluent turbidity meter. Thus, the effluent data during this time period was unreliable and omitted from the graph.

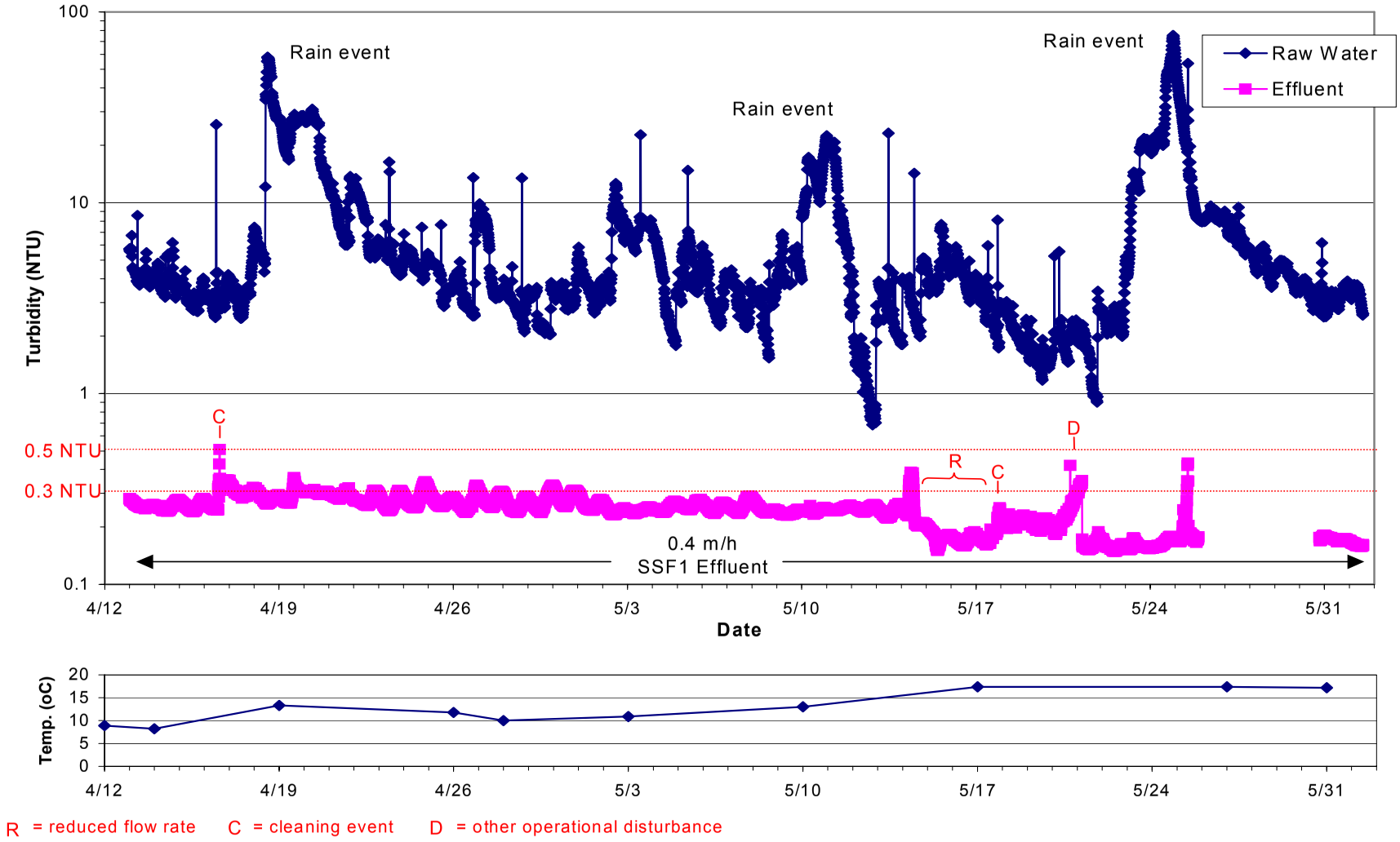

Figure 5.6: Pilot System 2 – Online Turbidity Data: April 13 – June 1, 2004

Performance during Warmer Water Conditions

During warmer water temperatures (8 to 17.5^{o}C) from Apr. 13 to June 1, with an average influent turbidity of 6.8 NTU and standard deviation of 8.5 NTU, and a hydraulic loading rate of 0.4 m/h, the slow sand filter 1 effluent, was below 0.5 NTU in 100% of the measurements, and below 0.3 NTU in 88% of the measurements. The average effluent turbidity of slow sand filter 1 was 0.25 NTU with a standard deviation of 0.05 NTU. It is recalled that this performance was measured after the multistage filter system had been in operation for 5 months. Thus, it was likely that increased biological maturation had an important role in producing such low effluent turbidity, in addition to increased biological treatment resulting from warmer water temperatures.

During warmer water conditions, it is evident that runoff events of high turbidity had little impact on effluent water quality. For example, on April 18 and May 11, during rain events with influent turbidities of 57 and 22.5 NTU, respectively, slow sand filter 1 effluent remained below 0.3 NTU on both occasions. Even during a rain event on May 24, with influent turbidity as high as 75 NTU, the spike in effluent turbidity was still below 0.5 NTU.

Operational disturbances, such as cleaning events, had little or no impact on slow sand filter effluent quality during warmer water conditions. For example, on Apr. 16, cleaning of the slow sand filter, by scraping the schmutzdecke, resulted in an increase of effluent turbidity from 0.25 NTU to only 0.51 NTU. In addition, on May 17, cleaning of the slow sand filter, also by scraping the schmutzdecke, resulted in virtually no effect on effluent quality. Finally, an operational disturbance of unknown origin occurred on May 20, and although it had a relatively larger effect on effluent quality, the effluent turbidity still remained below 0.5 NTU. Thus, during warmer water temperatures and with a more biologically mature system, the effluent had little or no sensitively to operational perturbations.

Summary of Results from Pilot System 2 (April 13 – June 1)

During warmer water temperatures (8 to 17.5^{o}C) from Apr. 13 to June 1, with an average influent turbidity of 6.8 NTU and standard deviation of 8.5 NTU, and a hydraulic loading rate of 0.4 m/h, the slow sand filter 1 effluent was below 0.5 NTU in 100% of the measurements,

and below 0.3 NTU in 88% of the measurements. The average effluent turbidity of slow sand filter 1 was 0.25 NTU with a standard deviation of 0.05 NTU.

This performance was achieved despite experiencing three major runoff events with influent turbidity as high as 75 NTU. In addition, operational disturbances, such as cleaning events, produced little or no effect on effluent turbidity. Clearly, increased biological maturity of the filter resulting from 5 months of filter operation, as well as increased biological treatment resulting from warmer water temperatures, resulted in a stable effluent quality that was less sensitive to large fluctuations in influent turbidity.

5.3.2.6 Comparison of Turbidity Performance between Pilot System 1 (Train 1) and Pilot System 2

The slow sand filter effluent turbidity of pilot system 1 (train 1) and pilot system 2 was compared during periods of similar hydraulic loadings in both cold and warm water conditions. As pilot system 1 (train 2) was tested at much higher hydraulic loading rates during the warmer water periods, its performance was not included in the comparison with pilot system 2. A comparison of effluent turbidity during colder water periods (<5^{o}C) of similar hydraulic loading was performed using data from Mar. 3. to Mar. 6 at 0.4 m/h, Mar. 9 to Mar. 15 at 0.2 m/h, and Mar. 15 to Mar. 19 at 0.4 m/h. A comparison of effluent turbidity during warmer water periods (~9 to 17.5^{o}C) of similar hydraulic loading was performed using data from Apr. 10 - Jun. 1 at a hydraulic loading of 0.4 m/h. Details of the statistical results of the comparison are shown in Appendix B (Table B.4).

During the colder water conditions, effluent measurements from pilot system 1 (train 1) were below 1 and 0.5 NTU in 92.7 and 86.5% of the measurements, respectively, which was similar to 94.3 and 82.2% in pilot system 2. With an average influent turbidity of 22.6 NTU (standard deviation of 25 NTU), the average effluent turbidity of pilot 1 (train 1) and pilot 2 was 0.44 and 0.5 NTU (standard deviation of 0.27 and 0.51 NTU), respectively. Based on a paired t-test at a 5% level of significance (1244 data points), this difference, although small, was statistically significant.

During the warmer water conditions, effluent measurements from pilot system 1 (train 1) were below 0.5 NTU in 99.1% of the measurements, which was similar to 100% in pilot system 2. However, effluent measurements were more frequently below 0.3 NTU in pilot system 1 (97.7%) compared to pilot system 2 (88%). In addition, with an average influent turbidity of 6.2 NTU (standard deviation of 8.4 NTU), the average effluent turbidity of pilot 1 (train 1) and pilot 2 was 0.19 and 0.24 NTU (standard deviation of 0.31 and 0.05 NTU), respectively. Based on a paired t-test at a 5% level of significance (2948 data points), this difference, although small, was statistically significant.

Although the difference in performance between both pilot systems was statistically significant, it was not a large difference. During the colder water conditions, the average effluent turbidity of pilot 2 was only 0.06 NTU higher than pilot 1 (train 1), and during the warmer conditions, the average effluent turbidity of pilot 2 was only 0.05 NTU higher. Nevertheless, pilot 2, due to its deeper bed depth in the roughing filter and slow sand filter, was expected to perform better than pilot 1. This did not in fact occur, which suggests that bed depth may not have been as important as was presumed.

Moreover, it is important to note that pilot system 1 had pre-ozonation whereas pilot system 2 did not. It is well known in the literature that pre-ozonation enhances the biodegradability of organic matter in the water, which leads to increased growth of biomass in the slow sand filter. This would result in improved treatment efficiency and reduced effluent turbidity. As shown previously, effluent measurements were more frequently below 0.3 NTU in pilot system 1, which had pre-ozonation, than in pilot system 2, which did not have pre-ozonation. Thus, it is possible that improved treatment efficiency resulting from pre-ozonation, compensated for any reduced treatment capacity resulting from shallower bed depths in pilot system 1. This resulted in overall similar performance from both systems, except during the warmer water conditions, where effluent turbidity from pilot 1 was more frequently below 0.3 NTU than pilot 2.

5.3.3 Operational Headloss Data and Filter Run Length

Hydraulic data (water levels) were collected from various locations throughout both pilot systems to monitor the development of headloss across the roughing filters and slow sand filters, and evaluate their filter run length. The filter run length is simply the amount of time elapsed between subsequent cleanings of the filter. Obviously, a longer filter run is desired for reducing maintenance requirements. One of the main benefits of slow sand filtration is its relatively long filter run (one or more months), compared to rapid sand filtration (one or two days or less). However, slow sand filtration suffers from dramatically reduced filter runs with high raw water turbidity that is consistently greater than 20 to 30 NTU (Wegelin, 1996). Thus, roughing filtration is used to remove and store solids before they enter the slow sand filter, hence limiting the development of headloss and increasing the filter run length of the slow sand filter. Obviously, this causes the roughing filter to require more frequent cleaning, instead of the slow sand filter, however it is considerably easier to clean a roughing filter, compared to a slow sand filter. For further details on cleaning the roughing filter and slow sand filter, refer to sections 2.5.4 and 2.3.4, respectively.

Water level data were obtained either from manually measuring the water level in the filter (as done in pilot system 1) or by measuring the water level in piezometers (as done pilot system 2). For details on the location of the piezometers, see Figure 3.10.

Figure 5.7 and Figure 5.8 shows the water level above the secondary ozone contactor and slow sand filter in pilot 1 (train 1) and pilot 1 (train 2), respectively, for the duration of the study. The water level in the secondary ozone contactor was important because the difference between the water level in the secondary ozone contactor and that above the slow sand filter was due to the headloss across the roughing filter (including approximately 5 cm headloss for piping). When the maximum water level in the secondary ozone contactor (~163.5 cm) was reached, the contactor would begin to overflow, thus reaching the end of the filter run for the roughing filter, at which time the roughing filter was cleaned. Likewise, the water level above the sand filter was important because when the maximum water level above the slow

sand filter (~150 cm) was reached, the slow sand filter would begin to overflow, thus reaching the end of the run for the slow sand filter, at which time it was cleaned. It is important to note that the water level above the sand filter influenced the water level in all previous stages, as water flowed through the system by gravity. For example, if the water level above the sand filter was just reaching its maximum, the water level in the secondary ozone contactor would already be backed up well beyond its maximum and overflowing to waste.

From the cleaning events indicated on both figures, it is evident that the frequency of cleaning was much higher for the roughing filter than the slow sand filter. In other words, the roughing filter had a much lower filter run length than the sand filter; however this was desirable because the roughing filter is generally much easier to clean than the slow sand filter. Most importantly, this indicates that the roughing filter fulfilled its purpose of increasing the filter run length of the slow sand filter by capturing and storing solids before entering the slow sand filter.

Figure 5.9 shows the headloss across each stage in pilot system 2. In this case, the headloss essentially represents the difference in the water level before and after the given stage in the filter. Again, it is evident that the cleaning events were much more frequent in the roughing filters than in the slow sand filters.

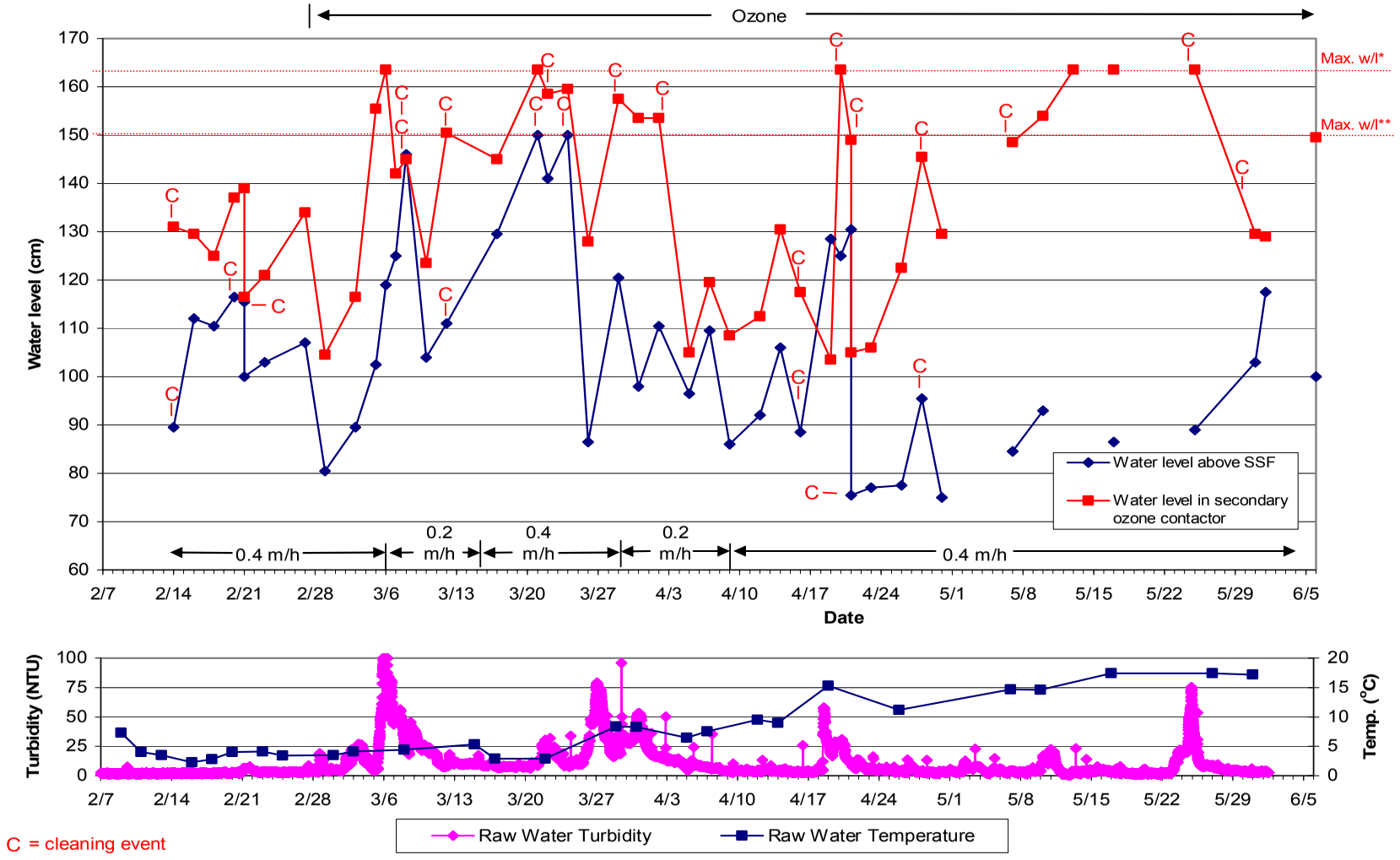

Figure 5.7: Hydraulic Data from Pilot System 1 (Train 1)

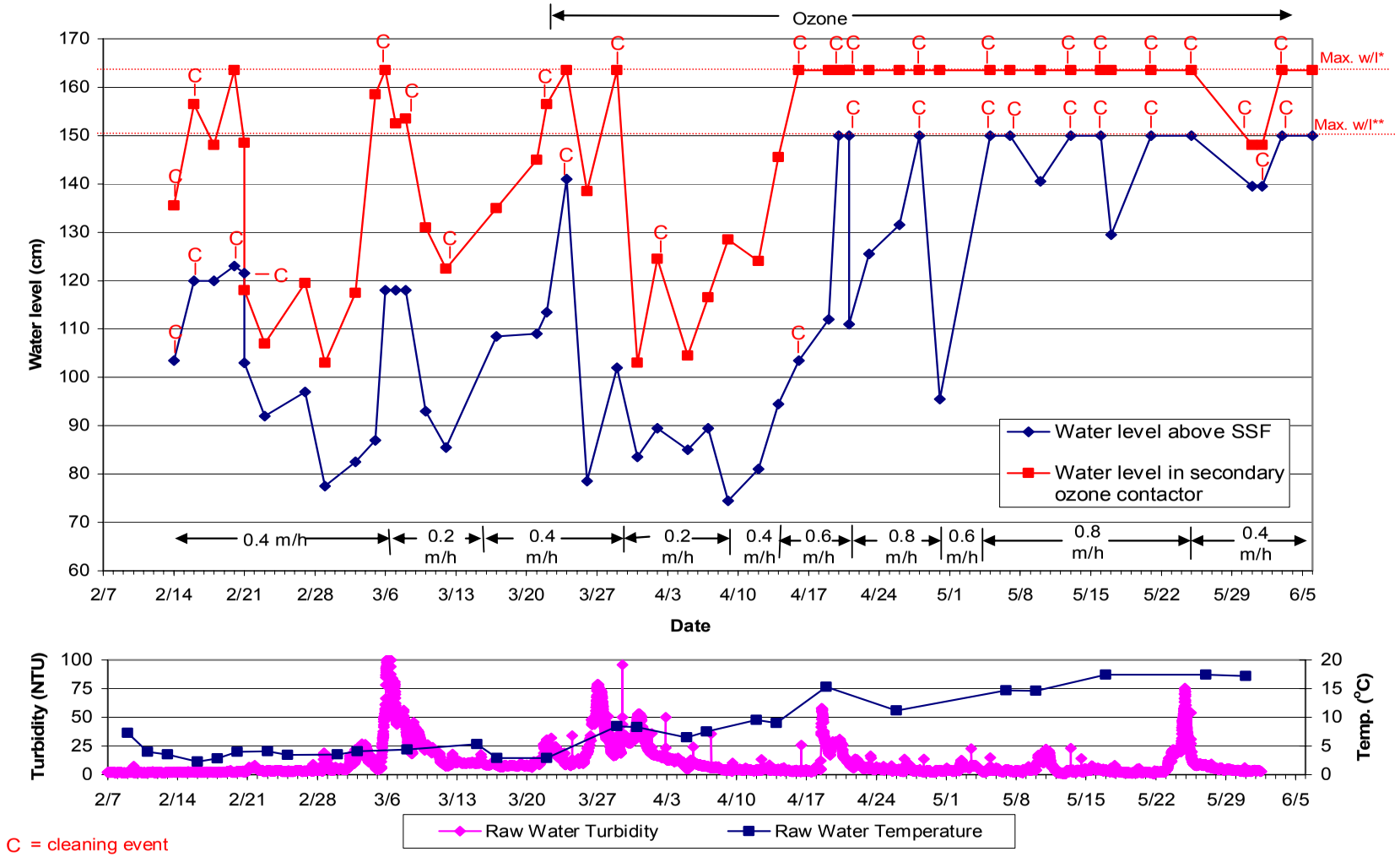

Figure 5.8: Hydraulic Data from Pilot System 1 (Train 2)

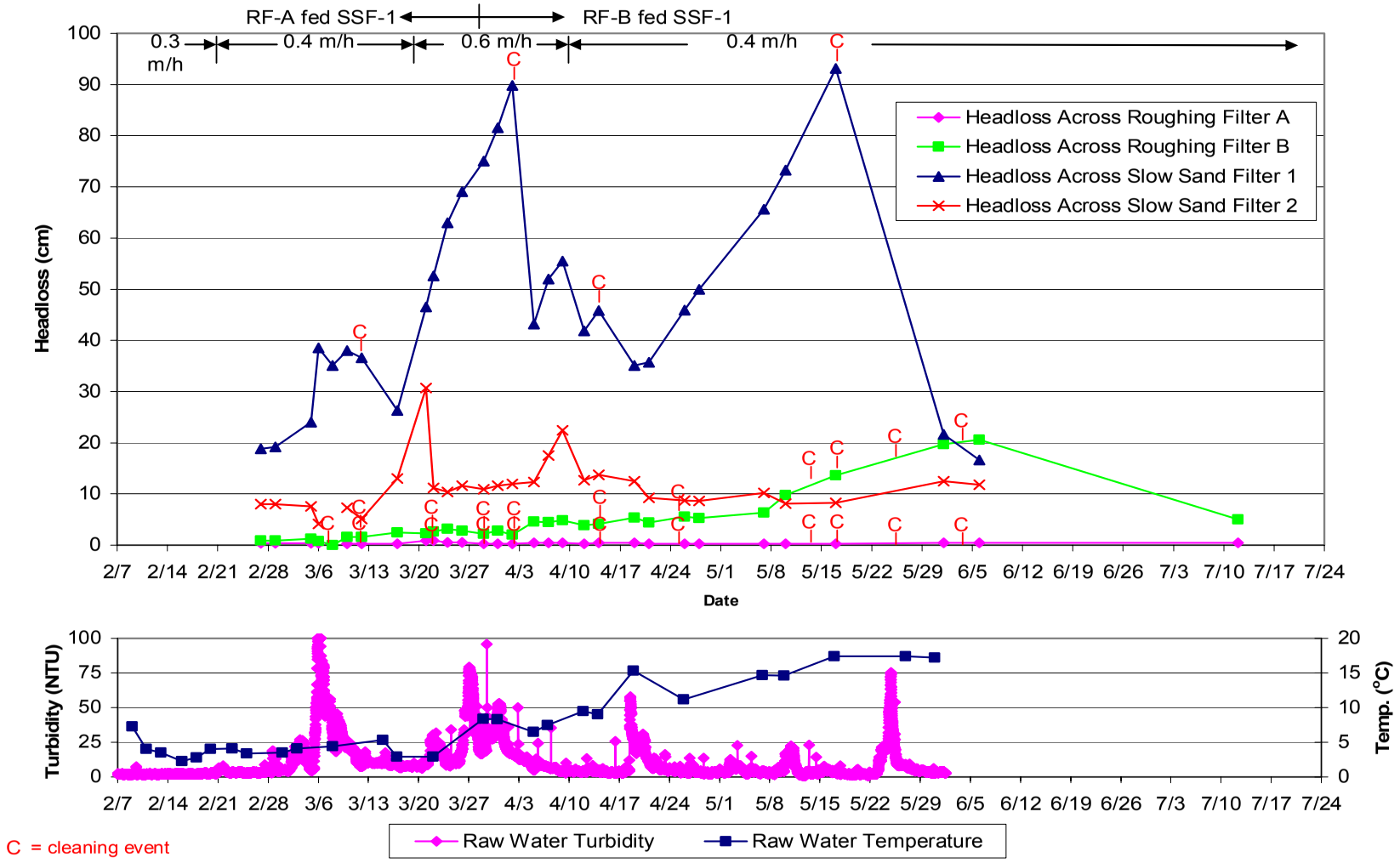

Figure 5.9: Hydraulic Data from Pilot System 2

For all the cleaning events shown in Figures 5.7 to 5.9, further data is provided in Table B.5 and B.6 in Appendix B for the roughing filter and slow sand filter, respectively. These data include the filter run length and hydraulic loading conditions between cleanings for the roughing filter and slow sand filter in both systems. Analysis of these data is discussed in the following sections.

5.3.3.1 Comparison of Roughing Filter Run Length in Pilot 1 and 2

The average filter run length of the roughing filters in pilot 1 was 6.4 days (standard deviation of 4.4 days) compared to 11.3 days (standard deviation of 9.5 days) in pilot 2. In other words, the filter run length of the roughing filters in pilot 2 was roughly double that of pilot 1. This is possibly due to the relatively higher hydraulic loading through the roughing filter in pilot 1 and its design, particularly with respect to the media size used in the filter. First of all, the pilot 1 roughing filters were designed to have a higher flow per unit area than that in pilot 2. For example, when both pilots were running at the same hydraulic loading rate through the slow sand filter, the hydraulic loading through the pilot 1 roughing filter was roughly double that of pilot 2. Throughout the study, the range of hydraulic loadings through the roughing filter in pilot 2 was 0.5 to 1.35 m/h (which corresponds to 0.2 to 0.6 m/h through the subsequent slow sand filter), compared to 0.75 to 3 m/h in pilot 2 (which corresponds to 0.2 to 0.8 m/h through the subsequent slow sand filter).

Wegelin et al. (1986) found that at increased filtration rates (2 m/h), coarse particles penetrated deeper into the bed and clogged the finer gravel media. This is because solids deposits are subjected to relatively high fluid shear forces which cause them to become re-suspended and deposited further into the finer gravel media, eventually clogging the finer media, and leading to more rapid development of headloss and decreased filter runs. Thus, the relatively higher hydraulic loading through the pilot 1 roughing filters, when compared to that in pilot 2, likely contributed to their having a much lower filter run length than in pilot 2.

The previously mentioned findings by Wegelin et al. (1986) also explain why the average filter run length in pilot 2 at a roughing filtration rate of 0.5 to 0.9 m/h was 13.2 days, compared to 5.5 days at 1.35 m/h. The difference in average filter run length between the

higher and lower hydraulic loadings was much less dramatic in pilot 1, where the average filter run length at 0.75 to 1.5 m/h was 6.9 days, compared to 4.9 days at 2.4 to 3 m/h. Perhaps this suggests that the hydraulic loading rate was not the only factor affecting the filter run length in the pilot 1 roughing filters or, perhaps the relationship of hydraulic loading versus run length may have been non-linear.

In the pilot 1 roughing filter, the bottom layer of gravel is a medium sized gravel that is approximately 8-12 mm in diameter. Generally, the bottom layer is most important for storing accumulations of solids that are removed from the raw water entering the bottom of the roughing filter and also solids that have previously been removed in the upper regions of the roughing filter, but are migrating down to the bottom under the influence of gravity. In pilot 1, the bottom layer of gravel was only 10 cm thick, thus it had limited storage capacity. Also, the second layer of gravel (2.4-3.4 mm in diameter) may have been too fine to provide adequate pore space for additional storage once the storage capacity of the bottom layer had been exhausted. Wegelin (1996) recommends a gravel layer ranging from 4-8 mm in size to follow the gravel layer ranging from 8-12 mm in size. Thus, the relatively large reduction in gravel size between the first and second layer of gravel may have been too abrupt, and once the solids storage capacity of the bottom layer was exhausted, solids accumulations may have begun to clog the much smaller pores of the second gravel layer. This may have led to a rapid headloss development in the second gravel layer, resulting in shorter filter runs.

As a comparison, roughing filter B in pilot 2 also had a 10 cm thick bottom layer of medium sized gravel, but it had a relatively coarser second layer of gravel (4.8-9.5 mm in diameter), compared to pilot 1, which likely had more pore space for substantial additional storage of solids after the storage capacity of the bottom layer had been exhausted. This likely contributed to a longer filter runs, compared to pilot 1.

Lastly, another potential factor that may have contributed to the shorter filter run length in pilot 1 is the reduced depth of the filter. The bed depth of the roughing in pilot 1 was only 0.6 m, compared to 1.2 m in pilot 2. According to Collins et al. (1994b), bed depth is the most influential design variable in roughing filters. As particles deposit in the filter bed, pore

spaces become smaller and the solid deposits are subjected to higher shear forces, causing detachment and penetration of detached solids deeper into the filter bed. Thus, it is important to maximize the bed depth to capture particles that penetrate deeper into the filter. In pilot 1, once the bed had reached its solids storage capacity, any solids that became re-suspended in the flow likely either exited the roughing filter and entered the slow sand filter, or may have accumulated on the top of the filter bed where interstitial velocity was lower, which would have likely lead to more rapid development of headloss and decreased filter run lengths.

Unfortunately, since pilot 1 and 2 did not follow the same routine of hydraulic loading conditions throughout the entire study, the difference in filter run length between the two pilots could not be proven statistically significant. Thus, further testing is required to determine quantitatively if the design of the roughing filter as well as the relatively higher hydraulic loadings had a negative impact on roughing filter run length in pilot 1.

5.3.3.2 Comparison of Slow Sand Filter Run Length in Pilot 1 and 2

The average filter run length of the slow sand filters in pilot 1 was 7.9 days (standard deviation of 7.1 days) compared to 37.3 days (standard deviation of 31.3 days) in pilot 2. Slow sand filtration generally has a reputation for long filter run lengths and requiring little maintenance, thus a filter run length of 8 days is a particularly poor filter run length when compared to the literature, which generally reports slow sand filter run lengths of one or more months. Again, because both pilots did not follow the same routine of hydraulic loading conditions, they could not be statistically compared. Nevertheless, there are a couple of hypotheses to explain the rather large difference in filter run length between the two pilots. First of all, the slow sand filter in pilot 1 experienced a period of time lasting 5 weeks when it was operated at a hydraulic loading ranging from 0.6 to 0.8 m/h, whereas pilot 2 only experienced an increased hydraulic loading of 0.6 m/h for 3 weeks. Despite the large difference in operational conditions between the two pilots, it is important to note that pilot 1 only averaged a filter run length of 8.8 days at a hydraulic loading of 0.2 to 0.4 m/h, compared to 5 days at a hydraulic loading of 0.6 to 0.8 m/h. In comparison, pilot 2 averaged a filter run length of 57 days at a hydraulic loading of 0.2 to 0.4 m/h, much larger than in pilot

1. Thus, it is unlikely that the large difference in filter run lengths between the two pilots can be explained solely by differences in operating conditions.

The following hypotheses describe potential factors that could have affected the filter run length in pilot 1, beyond just its operational conditions. Firstly, due to the limited bed depth and solids storage capacity of the pilot 1 roughing filter, solids may have been breaking through the filter and depositing in the slow sand filter, thus clogging pores in the fine sand and leading to increased development of headloss in the slow sand filter. Secondly, pre-ozonation, which generally increases the growth of biomass in the slow sand filter by rendering organics more biodegradable, was present in pilot system 1 and possibly increased the rate of headloss development due to greater growth of biomass. Thirdly, the method of cleaning had an impact on the ability to remove headloss causing material from the filter and maximize the filter run length until the next cleaning. Generally, near the beginning of the study, the slow sand filter was cleaned more frequently because the operator was unsuccessful in many attempts to simulate a wet harrow method of cleaning and headloss causing material was not sufficiently removed from the filter. The method of cleaning was then revised to a physical scraping of the schmutzdecke or an upflow of water through the slow sand filter, where solids where backwashed to waste. Both of these methods were effective in removing headloss causing material from the filter and relatively longer filter runs were achieved from the last week in February well in to April. Lastly, as shown at the bottom of Figures 5.7 to 5.9, there were a number of runoff events of high turbidity throughout the study that likely quickly exhausted the solids storage capacity of the roughing filter (which was lower than in pilot 2), leading to increased levels of turbidity entering the slow sand filter and resulting in clogging of the sand bed.

Finally, the effect of hydraulic loading rate on filter run length in both pilots was profound. At a given influent turbidity level, a higher hydraulic loading rate places a higher solids loading (flux of solids) onto the filter bed, leading to more rapid development of headloss. In addition, as mentioned in the previous section, a higher hydraulic loading rate through the roughing filter may cause solids deposits to become re-suspended and enter the slow sand filter. Thus, in late April and May, when the slow sand filter in pilot 1 (train 2) was operated

at a hydraulic loading of 0.6 to 0.8 m/h, it had a filter run length of only 5 days, and frequent cleaning was required, which is truly undesirable for slow sand filtration plants. Pilot system 2, which generally had much longer filter runs, had filter run lengths of only 14 to 21 days when operated at 0.6 m/h, which is still unacceptable for slow sand filtration. Clearly, due to the variable influent turbidity of the Grand River during this study, operating the filter at a hydraulic loading above 0.4 m/h led to drastic reductions in filter run lengths.

Practically speaking, a multistage system would likely only be operated at a higher hydraulic loading (>0.4 m/h) if one of two parallel systems was taken offline for cleaning, thus doubling the flow rate through the other system. Cleaning, generally would not take longer than a day before both systems were online again, thus filter run length would not be a concern for such a short lived surge in hydraulic loading.

Notwithstanding this, a higher hydraulic loading may be entirely suitable for a cleaner surface water with more predictable levels of turbidity, such that the filter could be operated at the higher hydraulic loading at the appropriate time of the year when raw water turbidity is lowest. This underlines the importance of pilot testing under specific site conditions to determine the optimal hydraulic loading for effluent quality and sufficient filter run lengths.

Overall, pre-ozonation, roughing filter design, the method of cleaning, increased hydraulic loadings, and increased influent turbidity likely all contributed to poor slow sand filter run lengths in pilot system 1. Pilot 2, which had no pre-ozonation, deeper roughing filters, and lower hydraulic loadings, had much longer slow sand filter run lengths. It is recommended that the roughing filter in pilot 1 be modified to a deeper bed depth for increased solids storage capacity and the filter area increased to reduce its range of hydraulic loadings. It is also recommended that the slow sand filter in pilot 1 not be operated at 0.8 m/h for more than one day, which is typically the time it takes to clean a parallel filter that has been taken offline.

5.3.3.3 Summary of Filter Run Length Results

In both pilot systems, the roughing filter had much shorter filter runs than the slow sand filter, requiring more frequent cleaning. As cleaning of the roughing is much easier than that of the slow sand filter, the roughing filter was essentially successful in preserving the filter run length of the slow sand filter by capturing a large amount of solids before entering the slow sand filter.

The filter run length of the roughing filters in pilot 1 was roughly half that in the pilot 2 roughing filters. This was likely because the roughing filters in pilot 1 experienced much higher hydraulic loadings than in pilot 2, which may have caused solid deposits to become re-suspended and deposit further into the finer gravel media, leading to more rapid development of headloss and decreased filter runs. In addition, the reduced depth and solids storage capacity of the first gravel layer, resulting in a rapid development of headloss in the second gravel layer may have also contributed to the reduced filter run length of the pilot 1 roughing filters. Furthermore, the overall bed depth of the pilot 1 roughing filters was roughly half that of the pilot 2 roughing filters, which may have led to larger solids accumulation per unit volume of filter media, resulting in higher headloss across the filter. Unfortunately, since pilot 1 and 2 did not follow the same routine of hydraulic loading conditions throughout the entire study, the difference in filter run length between the two pilots could not be proven statistically significant. Thus, further testing is required to determine if the design of the roughing filter as well as the relatively higher hydraulic loadings had a negative impact on the filter run length in the pilot 1 roughing filters.

The average filter run length of the slow sand filters in pilot 1 was 8 days, compared to 37 days in pilot 2. As slow sand filtration generally has a reputation for long filter run lengths and requiring little maintenance, a filter run length of 8 days was particularly poor. A potential cause of the poor filter run lengths in the pilot 1 slow sand filter were inadequate bed depth and solids storage capacity of the roughing filter, leading to solids breakthrough into the slow sand filter. Secondly, the relatively higher hydraulic loading rate through the roughing filter, compared to pilot 2, may have caused solids deposits to become re-suspended and enter the slow sand filter. Other potential factors were increased headloss due to pre-ozonation,

inadequate methods of cleaning and removing headloss from the filter, and increased hydraulic loadings through the slow sand filter, as well as a number of runoff events of high turbidity. Pilot 2, which had no pre-ozonation, deeper roughing filters, and lower hydraulic loadings, had much longer slow sand filter run lengths. It is recommended that the roughing filter in pilot 1 be modified to a deeper bed depth for increased solids storage capacity and the filter area increased to reduce its range of hydraulic loadings. It is also recommended that the slow sand filter in pilot 1 not be operated at 0.8 m/h for more than one day, which is typically the time it takes to clean a parallel filter that has been taken offline.

5.4 CONCLUSIONS

Pilot system 1, consisting of pre-ozonation, roughing filtration, slow sand filtration, and GAC filtration, was commissioned on October 7, 2003, and acclimatized for 4 months before testing was initiated on February 9, 2004. A decreasing trend in effluent turbidity was evident as the acclimatization process progressed. Pilot system 2, consisting of roughing filtration followed by two stages of slow sand filtration in series, was commissioned on December 19, 2003, and likely still undergoing acclimatization when testing initiated on February 9, 2004.

Specific conclusions of results from pilot system 1 and 2 during the various periods of testing in this study were summarized at the end of Sections 5.3.2.1 to 5.3.2.5 and in Section 5.3.3.3. A general listing of the overall findings from the online turbidity and filter run length results are given below.

5.4.1 Conclusions from Online Turbidity Results

5.4.1.1 Pilot System 1

1. In a less biologically mature system, with water temperatures ranging from <5 to 10^{o}C, an average influent turbidity of 14 NTU, a number of runoff events up to 100 NTU, and periods of hydraulic loadings as high as 0.4 m/h, pilot 1 multistage filter effluent was below 1 NTU in 92.7% and 90.1% of the measurements for train 1 and train 2, respectively, and below 0.5 NTU in 86.5% and 84.5% of the measurements. The average effluent turbidity in train 1 and 2 was 0.43 NTU and 0.45 NTU, with a standard deviation

of 0.41 and 1.6 NTU, respectively. During this phase of study, the filters were likely not fully biologically mature, and susceptible to reduced biological treatment in cold water conditions. Improved cold water performance is expected with a fully mature filter.

2. In a less mature system, performance during cold water temperatures (<5^{o}C) was particularly good during stable raw water quality conditions (influent turbidities below 10 NTU), hydraulic loadings, and operational conditions (no filter cleanings). During colder water temperatures, effluent measurements were below 0.5 NTU in 99.6% and 98.7% of the measurements for train 1 and 2, respectively, and below 0.3 NTU in 88.4% and 88.2% of the measurements. The average effluent turbidity of train 1 and 2 was 0.26 and 0.28 NTU (standard deviation of 0.17 and 2.24 NTU), respectively.

3. In a less mature system operating with cold water temperatures, a higher sensitivity of effluent turbidity to increased influent turbidity and operational disturbances (cleaning and temporary surges in hydraulic loading) resulted in effluent turbidity above 1 NTU on numerous occasions. This was likely due to inadequate biological maturity of the filter prior to the beginning of the study, decreased biological treatment due to cold water temperatures, relatively shallow bed depth (~0.5m) of the slow sand filter, and the relatively high hydraulic loading rate of 0.4 m/h through the slow sand filter (max. recommended in the literature). While a fully mature filter under similar operating conditions is expected to have improved performance, this remains to be determined.

4. In a less mature system operating with cold water temperatures (<10^{o}C), reducing the hydraulic loading rate of the slow sand filter to 0.2 m/h during runoff events of high turbidity (over approximately 25 NTU) was important for meeting the current Ontario standard of 1 NTU for slow sand filter effluent. It remains to be determined whether this would be required for a fully mature filter operating in cold water conditions. In warmer water temperatures and a more biologically mature filter, reducing the hydraulic loading rate during runoff events was not required.

5. In a more mature system operating with warm water conditions (7.5 to 17.5^{o}C), average

influent turbidity of 6.7 NTU, three major runoff events with influent turbidities ranging from 20 to 75 NTU, and hydraulic loading rates ranging from 0.4 m/h (train 1) to 0.8 m/h (train 2), the multistage filter effluent was below 0.5 NTU in 99.1% and 98.5% of the measurements for train 1 and train 2, respectively, and below 0.3 NTU in 97.7% and 87.9% of measurements. The average effluent turbidity of train 1 and 2 was 0.17 NTU and 0.23 NTU, with a standard deviation of 0.25 and 0.1 NTU, respectively. This performance was likely attributable to increased biological maturity of the filter, and increased water temperature and subsequent biological treatment, resulting in lower sensitivity of effluent turbidity to large fluctuations in influent turbidity. It remains to be determined whether similar performance at a hydraulic loading rate of 0.8 m/h can be achieved in a fully biologically mature filter at lower water temperatures (<5^{o}C).

6. There was no evidence that the granular activated carbon filter (GAC) filter provided additional removal of turbidity beyond that achieved by the slow sand filter. However, the filter was not fully commissioned at the time of testing and was occasionally producing turbidity via the escape of GAC fines.

7. The presence of pre-ozonation had no significant effect on effluent turbidity.

5.4.1.2 Pilot System 2

1. In a less biologically mature system, with cold water temperatures (<5^{o}C), average influent turbidity of 22 NTU, a runoff event up to 100 NTU, and hydraulic loading rates ranging from 0.2 to 0.4 m/h, the effluent measurements were below 1 NTU in 94.3% of the measurements, and below 0.5 NTU in 82.2% of the measurements (this analysis is conservative as it includes a period when effluent from slow sand filter 1 was measured, and a period when effluent from slow sand filter 2 was measured). The average effluent turbidity was 0.46 NTU, with a standard deviation of 0.5 NTU. During this phase of study, the filters were not fully biologically mature, thus susceptible to reduced biological treatment in cold water conditions.

2. Although the pilot 2 multistage filter, in absence of the second slow sand filter, performed well at lower influent turbidities and water temperatures below 5^{o}C, it failed to produce effluent below 1 NTU during runoff events of high turbidity when operating at a hydraulic loading rate of 0.6 /h. This was likely partly due to inadequate biological maturation of the filter, inadequate acclimatization of the filter to the increased organic/nutrient loading conditions, decreased biological treatment at lower water temperatures, and the high hydraulic loading rate. It remains to be seen whether a biologically mature pilot system 2, under similar operational conditions and in the absence of a second slow sand filter in series, could achieve effluent below 1 NTU.

3. In a less mature system, the second slow sand filter in series proved to be important for providing additional robustness to the process by reducing effluent turbidity to below 1 NTU during a couple of cold water runoff events of high turbidity combined with a relatively high hydraulic loading (0.4 m/h). Alternatively, in the absence of the second slow sand filter, reducing the hydraulic loading rate to 0.2 m/h was another method of achieving an effluent below 1 NTU during these challenging conditions.

4. With water temperatures ranging from 5 to 10^{o}C, average influent turbidity of 23 NTU, three runoff events ranging 30 to 80 NTU, and hydraulic loading rate of 0.6 m/h, the effluent from slow sand filter 1 was below 1 NTU in 82.2% of the measurements, and below 0.5 NTU in only 13.1% of the measurements. The average effluent turbidity was 0.78 NTU with a standard deviation of 0.27 NTU. During this phase of study, it was likely that the filters were still not fully mature.

5. Operating the multistage filter at a hydraulic loading rate of 0.6 m/h through the slow sand filter resulted in an effluent turbidity above 1 NTU during runoff events of high turbidity (25 to 75 NTU) and filter cleaning events (cleaning of the slow sand filter). In absence of the second slow filter in series, pilot 2 could only consistently produce an effluent below 1 NTU when raw water turbidity was less than 14 NTU. Performance would have likely been improved had the filters been fully acclimatized to the increased organic/nutrient loading conditions.

6. At a hydraulic loading rate of 0.6 m/h, the second slow sand filter was important for providing additional removal of turbidity beyond that achieved by slow sand filter 1. Its impact would have likely been more profound during periods of higher influent turbidity; however this remains to be determined from the handheld turbidity results in Chapter 6.

7. In a more mature system operating with warmer water temperatures (8 to 17.5^{o}C), average influent turbidity of 6.8 NTU, three runoff events up to 75 NTU, and hydraulic loading rate of 0.4 m/h, slow sand filter 1 effluent was below 0.5 NTU in 100% of the measurements, and below 0.3 NTU in 88% of the measurements. The average effluent turbidity of slow sand filter 1 was 0.25 NTU with a standard deviation of 0.05 NTU. With warmer water temperatures and a more biologically mature filter, performance was improved and the effluent was less sensitive to large fluctuations in influent turbidity and operational disturbances, such as filter cleaning events, which produced little or no effect on effluent turbidity.

5.4.1.3 Comparison of Pilot System 1 and 2

1. Although the slow sand filter bed depth of pilot system 1 (0.5 m) was much shallower than pilot system 2 (1 m), the performance of both pilot systems was generally similar throughout the study, in terms of average effluent turbidity and the frequency in which effluent measurements were below regulatory standards such as 1 NTU and 0.5 NTU.

2. It was believed that improved treatment efficiency of pilot system 1 due to enhanced biological growth resulting from pre-ozonation, may have compensated for the potential of reduced treatment capacity resulting from its shallower bed depths. This would explain the overall similar treatment performance between pilot system 1 and 2 (despite pilot 2 having twice the bed depth as pilot 1), except during warmer water conditions, where effluent turbidity from pilot 1, which had pre-ozonation, was more frequently below 0.3 NTU than in pilot 2, which did not have pre-ozonation.

3. Both pilot systems experienced a high sensitivity of effluent turbidity to increased influent turbidity during cold water conditions when the systems were less biologically mature. However, during warmer conditions when the systems were more mature, the effluent from both systems had little or no sensitivity to large fluctuations in influent turbidity.

5.4.2 Conclusions from Filter Run Length Results

1. The roughing filter was successful in preserving the filter run length of the slow sand filter by capturing a large amount of solids before entering the slow sand filter. This resulted in significantly shorter filter runs in the roughing filter than that of the slow sand filter, however this was desirable as cleaning of the roughing filter is much easier than cleaning of the slow sand filter.

2. The filter run length of the pilot 1 roughing filters was roughly half that of the pilot 2 roughing filters, likely due to its higher hydraulic loading (smaller filter area), and reduced bed depth and solids storage capacity. In addition, the reduced depth and solids storage capacity of the first gravel layer, resulting in a rapid development of headloss in the second gravel layer, which may have been too fine a media, may have also contributed to the reduced filter run length of the pilot 1 roughing filters. As both pilots did not follow the same routine of hydraulic loading conditions throughout the entire study, the difference in filter run length between the two pilots could not be proven statistically significant. Thus, further testing is required to determine if the design of the roughing filter as well as the relatively higher hydraulic loadings had a negative impact on the filter run length of the pilot 1 roughing filters.

3. The average filter run length of the slow sand filter in pilot 1 was 8 days, compared to 37 days in pilot 2. A potential cause of poor filter run lengths in the pilot 1 slow sand filter was an inadequate bed depth and solids storage capacity in the pre-roughing filtration stage, leading to solids breakthrough into the slow sand filter. Secondly, the relatively higher hydraulic loading rate through the roughing filter, which had a smaller filter area compared to pilot 2, may have caused solids deposits to become re-suspended and

breakthrough into the slow sand filter. Other potential factors were increased headloss due to pre-ozonation, inadequate methods of cleaning and removal of headloss causing material from the filter, and increased hydraulic loadings through the slow sand filter (0.8 m/h), as well as a number of runoff events of high turbidity.

4. Pilot 2, which had no pre-ozonation, deeper roughing filters, and was not operated at 0.8 m/h, generally had much longer slow sand filter run lengths. It is recommended that the roughing filter in pilot 1 be modified to a deeper bed depth for increased solids storage capacity and the filter area increased to reduce its range of hydraulic loadings. It is also recommended that the slow sand filter in pilot 1 not be operated at 0.8 m/h for more than one day, which is typically the time it takes to clean a parallel filter that has been taken offline.

Chapter 6

6 Performance of Multistage Filtration: Results from Manual Turbidity, Total and Fecal Coliforms, and *Cryptosporidium* Challenge Tests

6.1 INTRODUCTION AND OBJECTIVES

As described previously, the primary objective of this study was to investigate the potential of multistage filtration technology as a reliable and robust drinking water treatment process to treat surface water for small and non-municipal water systems in northern climates, where seasonal fluctuations in surface water turbidity and temperature pose challenging conditions from a treatment standpoint. The detailed objectives for the portion of the study described in this chapter were to determine:

1. The efficiency of multistage filtration in removing turbidity and coliform bacteria at higher raw water turbidity levels (over 50 NTU), low temperature (<5^{o}C), and higher hydraulic loading rates (up to 0.8 m/h).
2. The removal of *Cryptosporidium* oocysts by challenging the slow sand filter with influent concentrations of 10^{6} oocysts/L at higher hydraulic loading rates (0.4 to 0.8 m/h).
3. The importance of the roughing filter as a significant treatment barrier in the overall process of pathogen removal, beyond its traditional role of protecting the operational integrity of the slow sand filter.
4. The impact of pre-ozonation on the removal of coliform bacteria.
5. The impact and added robustness of an additional stage of slow sand filtration.

6.2 MATERIALS AND METHODS

The results described in this chapter were obtained during the same period discussed in Chapter 5. The reader is referred to Section 5.2 for a summary description of the pilot facilities and testing methods during that time. A full description of the pilot facilities is given in Chapter 3.

For the investigations discussed in this chapter, water samples were collected from various locations throughout each process and analyzed onsite and/or at the University of Waterloo laboratory for turbidity, total coliforms and fecal coliforms. In addition, the slow sand filters were challenged with high concentrations of formalin-inactivated *Cryptosporidium parvum* oocysts (10^6 oocysts/L) to determine removal efficiency. More details on sampling locations and frequency are given in Chapter 3.

Manual (handheld) turbidity measurements were obtained 2 to 3 times per week using a portable HACH 2100P turbidimeter. Continuous turbidity measurements were also obtained on raw water and final effluent using online HACH 1720D turbidity meters. Continuous turbidity data is not discussed in this chapter because it only indicates performance of the entire process, and does not reveal removals in each stage of the process.

In general, there was good agreement between the manual and continuous turbidity data for the final effluent in pilot system 1 and 2 (see Appendix C: Table C.2). The manual turbidimeter measurements were only 0.01 to 0.025 NTU higher than the online turbidimeter measurements. Although some small difference between individual instruments will always exist, this difference may be attributed to imperfections in the glass of the sample vial for the manual turbidimeter, which generally contributes to background turbidity. In fact, when the vial was coated with oil, as recommended by the manufacturer, the turbidity of a sample of ultra-pure water dropped from 0.06 to 0.04 NTU, a difference of 0.02 NTU, which explains the difference between manual and online turbidity measurements.

Total coliforms were analyzed according to Standard Methods, Method 9221: Multiple Tube Fermentation Technique; and fecal coliforms were analyzed according to Method 9222: Fecal Coliform Membrane Filter Procedure. For total coliforms, positive tubes were scored using the most probable number (MPN) Table 9221.IV in Standard Methods and reported as MPN/100 mL. The detection limit was <2 MPN/100 mL. For results that were <2 MPN/100 mL, log removals were calculated by using 2 MPN/100 mL as the effluent concentration. This resulted in calculated log removals that were conservative. The resulting log removals were reported as "greater than" the calculated removal (i.e. >2 logs removal). On the graphed results, a result of <2 MPN/100 mL is represented by showing 1 MPN/100 mL as a "dummy" number.

The fecal coliform method had a detection limit of 0 CFU/100 mL. However, to calculate log removals for results of 0 CFU/100 mL, the concentration of 0.1 CFU/100 mL was used. The resulting log removals were therefore understated, and were reported as '"greater than" the calculated removal. Due to the log scale on the graphed results, a result of 0 CFU/100 mL is shown as 0.1 CFU/100 mL.

For the *Cryptosporidium* challenge tests, the slow sand filters were continuously seeded with a feedstock of 10^6 oocysts/L of formalin-inactivated *Cryptosporidium parvum* oocysts. The feedstock was fed into the slow sand filter influent for a duration of 6 hours to obtain as closely as possible a steady state effluent concentration for quantifying removals. *Cryptosporidium parvum* oocysts were measured using Method 1622/1623: IFA staining method (USEPA, 2001), which involved filtration of the water sample through a membrane, staining with fluorescent antibodies, followed by microscopic enumeration.

Further details of the turbidity, total coliform, fecal coliform and *Cryptosporidium* methods are given in Chapter 3.

6.3 RESULTS AND DISCUSSION

6.3.1 Manual Turbidity

This section analyzes and discusses the removal of turbidity in each stage of the multistage filter system, as well as the cumulative removal in the entire process. The raw data used to generate the graphs shown in this section can be found in Appendix C (Table C.3 to C.5). Turbidity data was collected 2 to 3 times per week for a total of approximately 30 data points at each sampling location in the process. Note that results are presented initially for the entire period, as opposed to Chapter 5, where results were presented for sub-periods.

Runoff events and hydraulic loading conditions for the slow sand filters are shown on the graphs, and influent water temperatures are indicated below each graph. Turbidity is shown on a logarithmic scale on all graphs. The current Ontario drinking water turbidity standard of 0.5 NTU for conventional treatment (MOE, 2003) is also shown, as well as the future proposed Canadian turbidity guideline of 0.3 NTU (a new Canadian guideline would likely be adopted as a standard by Ontario). It is important to note, however, that the current Ontario effluent turbidity standard for slow sand filtration is 1 NTU (MOE, 2003).

6.3.1.1 Pilot System 1 (Train 1 and Train 2)

Figure 6.1 and Figure 6.2 show the influent turbidity and the turbidity after each stage in trains 1 and 2, respectively. Figure 6.3 and Figure 6.4 shows the log removal of turbidity in each stage as well as the total cumulative removal in the entire process. Over the 4-month testing period, different slow sand filter hydraulic loading rates (filtration rates) were tested, ranging from 0.2 to 0.4 m/h, and as high as 0.8 m/h (train 2). (See Table 3.3 in Chapter 3 for the corresponding roughing filter hydraulic loading rates.) In the figures, any missing data for the slow sand filter effluent occurred because the filter was overflowing and had a much lower hydraulic loading rate than indicated, resulting in low and unrepresentative effluent turbidity. The GAC filter effluent was only sampled throughout March and April.

While log removals may be useful information for evaluating filter performance, it is important to note that the calculated log removals of turbidity were influenced by the level of influent turbidity (i.e. higher removals occurred during runoff events when influent turbidities were higher). Thus, it is more accurate to evaluate filter performance based on effluent turbidity levels. This is especially important for evaluating slow sand filter performance, as the turbidity of slow sand filter influent is substantially reduced by pre-roughing filtration, thus calculated log removals through the slow sand filter would be reduced.

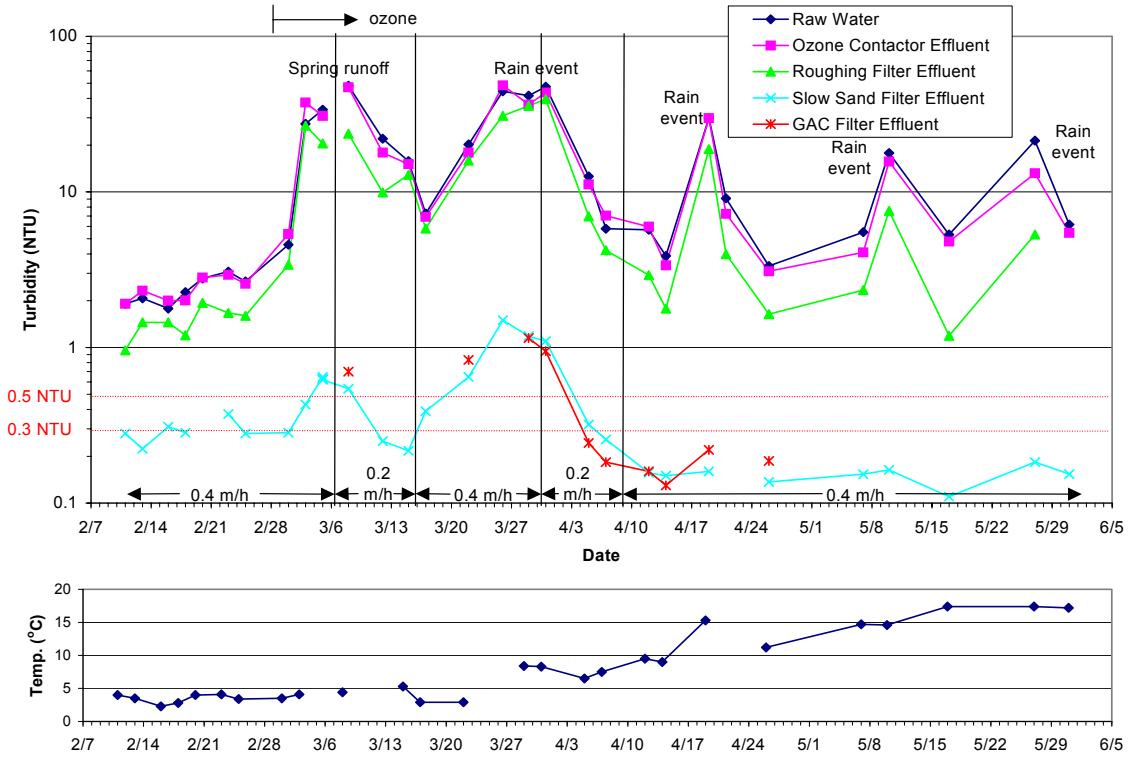

Figure 6.1: Pilot System 1 (Train 1) – Turbidity after each stage

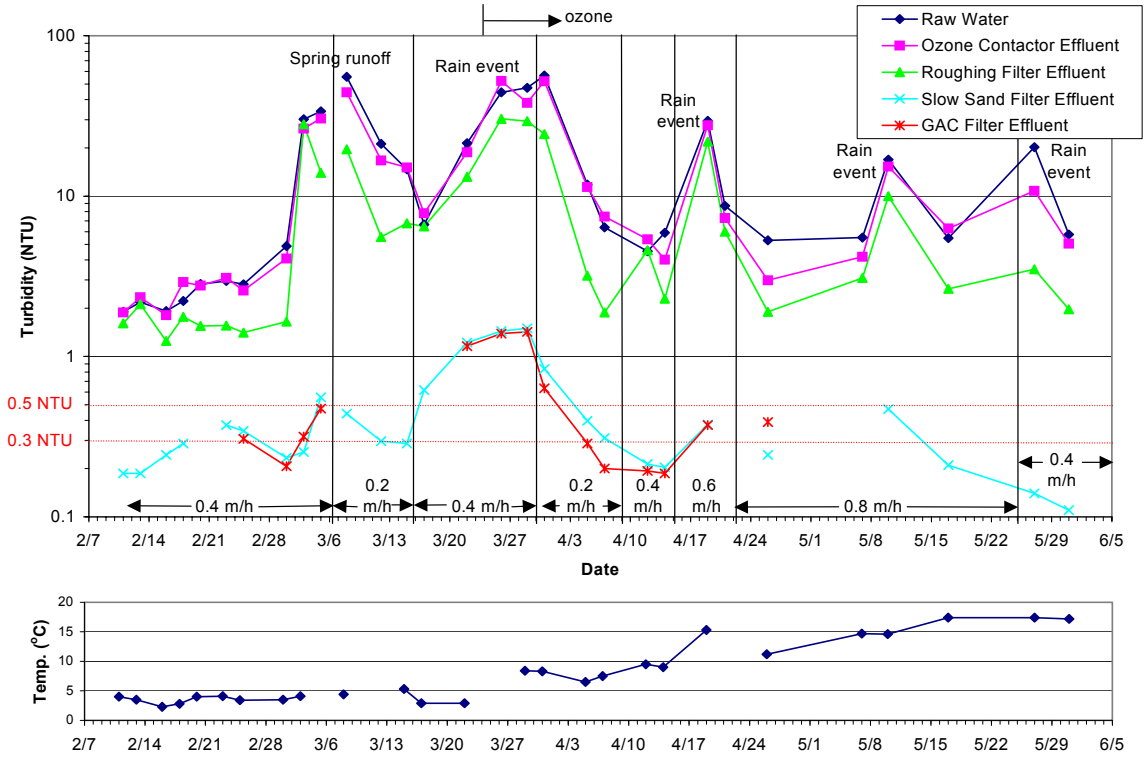

Figure 6.2: Pilot System 1 (Train 2) – Turbidity after each stage

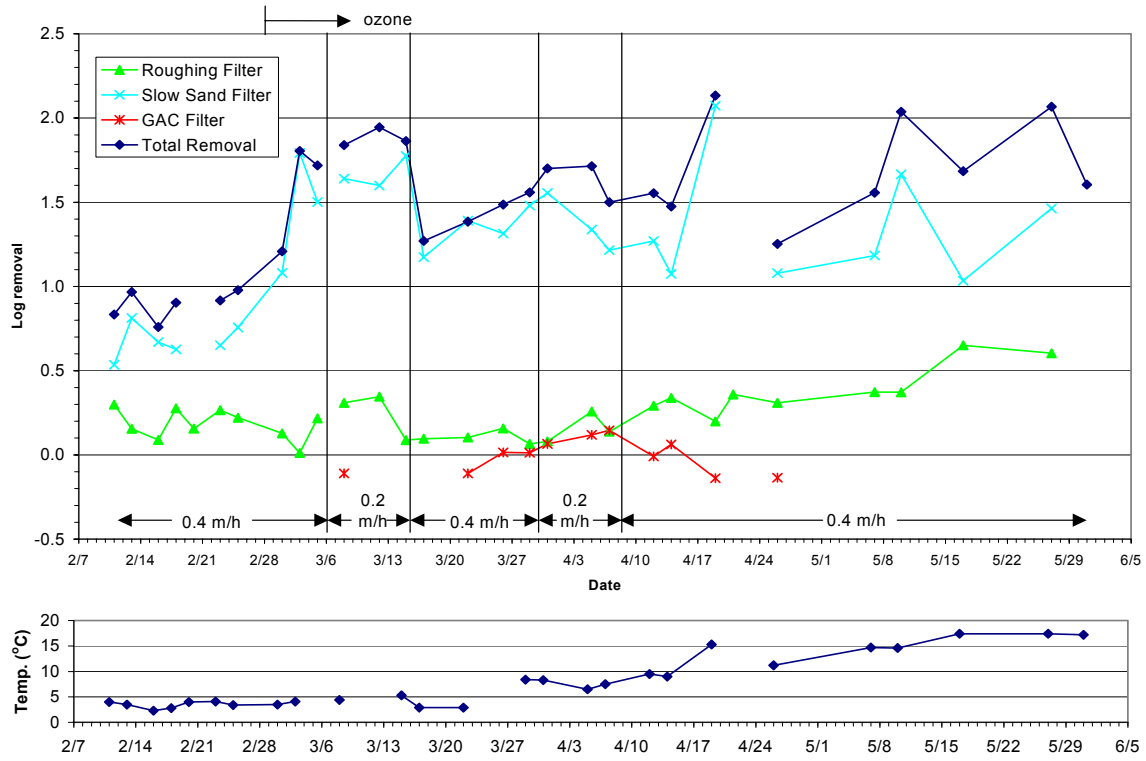

Figure 6.3: Pilot System 1 (Train 1) – Log removal of turbidity in each stage

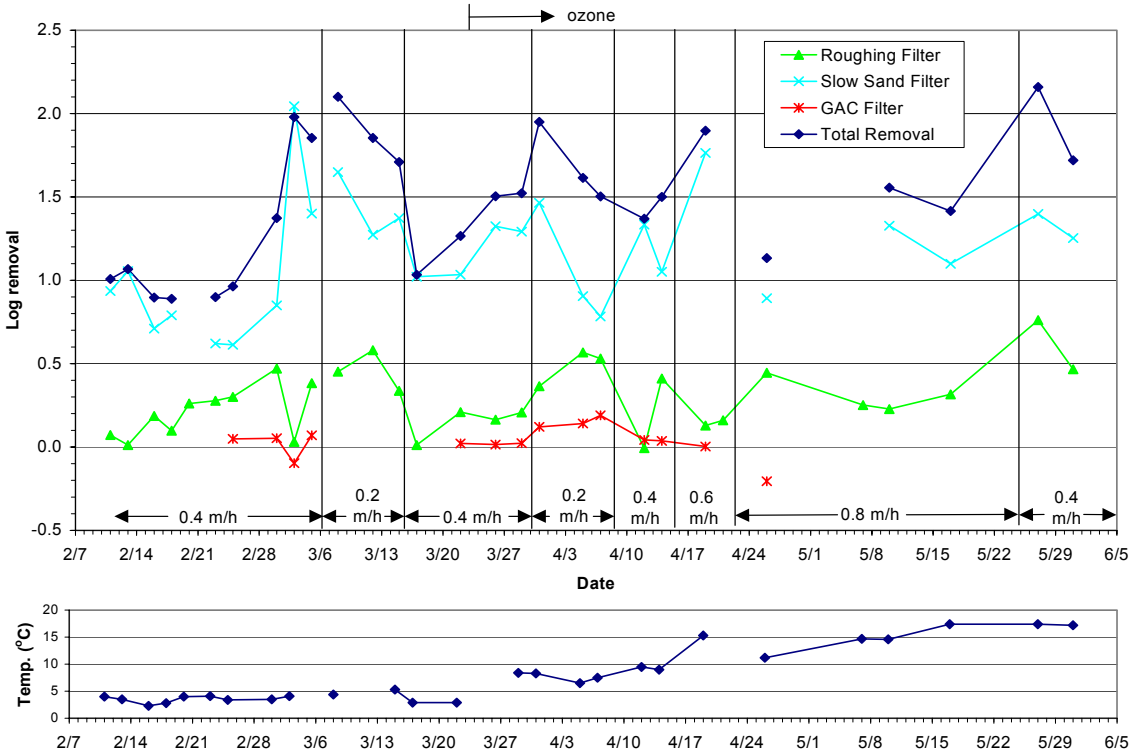

Figure 6.4: Pilot System 1 (Train 2) – Log removal of turbidity in each stage

Performance During Colder Water Temperatures

From Feb. 11 to Mar. 17, the slow sand filter effluent in both trains was consistently below 1 NTU, despite an average influent turbidity of 12 NTU and a spring runoff event reaching approximately 50 NTU, temperatures below 5 ^{o}C, and slow sand filtration rates as high as 0.4 m/h. Furthermore, 86% of the slow sand filter effluent measurements were below 0.5 NTU. The roughing filters reduced influent turbidities to an average of 8.1 NTU and 6.7 NTU in train 1 and 2, respectively. The average slow sand filter effluent turbidity was 0.38 NTU (standard deviation of 0.15) and 0.33 NTU (standard deviation of 0.13) in train 1 and 2, respectively. This performance was impressive, especially considering that the filters had only been in operation for a few months in cold water conditions, thus the filters were likely not fully biologically mature at the beginning of the study.

Removal of turbidity in the train 1 roughing filter averaged 0.19 logs during colder water conditions, and reached a maximum of 0.4 logs. Turbidity removals were similar in train 2, which averaged 0.25 logs, and reached a maximum of 0.6 logs removal. Turbidity removals

in the slow sand filter averaged 1.1 logs and ranged from 0.6 to 2 logs in train 2, compared to an average of 1.12 logs ranging from 0.5 to 1.8 logs in train 1. Finally, cumulative turbidity removals after the slow sand filter averaged 1.35 logs and ranged from 0.9 to 2.1 logs in train 2, compared to an average of 1.32 logs ranging from 0.8 to 1.9 logs in train 1.

Performance During Warmer Water Temperatures and Increased Hydraulic Loading

From Apr. 19 to May 31, the water temperature was consistently above 10^{o}C and increased throughout the spring season. During this time, train 1 consistently produced effluent below 0.3 NTU, despite an average influent turbidity of 12 NTU and three runoff events with influent turbidities of 30, 18, and 21 NTU, and a slow sand filtration rate of 0.4 m/h. Train 2 consistently produced effluent below 0.5 NTU at 0.8 m/h, which is remarkable for such a high slow sand filtration rate. There was not enough handheld turbidity sample data to determine if the increased hydraulic loading rate had a significant effect on effluent turbidity (see Appendix C: Table C.6). However, it was determined from online turbidity results in Chapter 5 that the increased hydraulic loading of 0.8 m/h had a negative impact on effluent turbidity, however most effluent measurements were still below 0.5 NTU, likely due to increased biological maturity of the filter and increased water temperatures. From the handheld turbidity measurements, it was also evident that filter performance was improved in warmer water conditions and with increased filter maturity.

The roughing filters reduced influent turbidities to an average of 5.9 NTU and 6.4 NTU in train 1 and 2, respectively. The average slow sand filter effluent turbidity was 0.15 NTU (standard deviation of 0.02) and 0.26 NTU (standard deviation of 0.14) in train 1 and 2, respectively. Although train 2 achieved good effluent turbidity at 0.8 m/h, it remains to be seen whether this can also be achieved in colder water temperatures with a fully mature filter.

Removal of turbidity in the train 1 roughing filter averaged 0.41 logs during warmer water conditions, and reached a maximum of 0.65 logs. Turbidity removals were similar in train 2, which averaged 0.34 logs, and reached a maximum of 0.76 logs removal. Turbidity removals in the slow sand filter averaged 1.29 logs and ranged from 0.9 to 1.8 logs in train 2, compared to an average of 1.42 logs ranging from 1 to 2.1 logs in train 1. Finally, cumulative turbidity

removals after the slow sand filter averaged 1.7 logs and ranged from 1.3 to 2.2 logs in train 2, compared to an average of 1.8 logs ranging from 1.4 to 2.3 logs in train 1.

Comparison of Performance during Cold and Warm Water Temperatures

Before results from warm and cold water temperatures could be statistically compared, it was important to determine whether the influent conditions were similar during these two time periods, as effluent turbidity levels and filter removal rates are dependent on the influent turbidity. Based on a two-tailed t-test with a 5% level of significance (see Appendix C: Table C.7 to C.8), the influent turbidity during cold water conditions (Feb. 11 to Mar. 17) was statistically similar to that during warm water conditions (Apr. 19 to May 31). Thus, the effluent results during warm and cold water conditions can be reliably compared.

In train 1, the average slow sand filter effluent turbidity was 0.38 NTU in cold water conditions, compared to 0.15 NTU in warm water conditions. Based on a two-tailed t-test at a 5% level of significance, the slow sand filter effluent turbidity in train 1 was significantly lower in warmer water temperatures. The same statistical comparison could not reliably be applied to train 2 because it operated at a higher hydraulic loading rate for much of the time during the warm water conditions.

Removal of turbidity in the roughing filter of train 1 averaged 0.41 logs during warm water temperatures, compared to 0.19 logs during colder water temperatures. This difference was determined to be significant based on a two-tailed t-test at a 5% level of significance (see Appendix C: Table C.7 to C.8). Removal of turbidity in the slow sand filter of train 1 averaged 1.42 logs during warmer water temperatures, which was not statistically higher than an average of 1.12 logs removal during colder water temperatures. Finally, the cumulative removal of turbidity in train 1 averaged 1.8 logs in warm water, compared to 1.32 logs in cold water. Based on a two-tailed t-test at a 5% level of significance, the cumulative removal in train 1 in warm water was higher than that during colder water temperatures.

Thus, multistage filter performance did improve during warmer water temperatures, but was likely due to a combination of increased biological activity in warmer temperatures and increased filter maturity.

The Effect of Lower Hydraulic Loading Rate during Runoff Events

From the online turbidity results in Chapter 5, it was determined that reducing the hydraulic loading rate during runoff events was important for maintaining effluent below 1 NTU in a less mature system. From the handheld turbidity measurements, the effect of reducing the hydraulic loading rate during runoff events was again shown during a spring runoff on Mar. 8, when an influent turbidity of approximately 50 NTU was reduced to an effluent value of 0.54 NTU in train 1, and 0.44 in train 2. This was achieved with a slow sand filtration rate of 0.2 m/h. Interestingly, the previous 3 measurements prior to Mar. 8 displayed an increasing effluent trend with increasing influent turbidity, with a slow sand filtration rate of 0.4 m/h. However, after the filtration rate was reduced to 0.2 m/h, this trend was disrupted and the multistage filter produced an effluent turbidity lower than expected, despite a continued increase in influent turbidity.

On Mar. 22 to Mar. 29, during a runoff event with influent turbidities reaching approximately 50 NTU, both train 1 and 2 (operated at a slow sand filtration rate of 0.4 m/h) produced effluent as high as 1.5 NTU, exceeding the Ontario slow sand filter limit of 1 NTU. Based on the results from the Mar. 8 runoff event, it is expected that a lower filtration rate would have produced an effluent lower than 1 NTU during this event. It is also probable that lower effluent turbidity would have been obtained if the filters were fully mature.

In addition, the roughing filters in train 1 and train 2 only contributed up to 0.16 logs and 0.21 logs removal, respectively. It is possible that the roughing filters had exceeded their solids storage capacity during this high turbidity event. It is expected that roughing filters with a deeper bed depth would have attenuated more of the solids loading on the slow sand filter and resulted in lower effluent turbidities. However, roughing filter A in pilot system 2 had twice the bed depth of the roughing filters in pilot system 1, and removal in the roughing filters on this date was approximately the same. In addition, the roughing filters in both pilot systems

had approximately the same hydraulic loading rate, 1.5 m/h in pilot 1 and 1.35 m/h in pilot 2. Thus, it is possible that the decreased removals could have been due to a high proportion of colloidal matter in the influent. This was especially suspected as the cause of poor effluent turbidity on Mar. 22. Interestingly, as discussed in the Section 6.3.2, removal of total and fecal coliforms was also uncharacteristically poor on Mar. 22.

The Effect of Pre-ozonation on Effluent Turbidity

Train 1 operated with pre-ozonation from Feb. 23 to Mar. 22, while train 2 operated without pre-ozonation until Mar. 22. A paired t-test was performed on the effluent turbidity results from train 1 (with ozone) and train 2 (no ozone) to determine whether pre-ozonation had a significant effect of effluent turbidity. Details of the paired t-test are shown in Appendix C (Table C.11). At a 5% level of significance (two-tailed test), there was not a significant difference in the results between train 1 and train 2. Thus, pre-ozonation did not have a significant effect on effluent turbidity during this time period. However, this was based on a relatively limited data set taken over a short period of time. Pre-ozonation might have an effect on effluent turbidity if analyzed over a longer time period or during different conditions (e.g. during warmer water conditions).

The Effect of Post Treatment with GAC Filtration

Occasionally, the granular activated carbon (GAC) filter contributed a small removal of turbidity beyond that provided by the slow sand filter. In train 1, on Mar. 31, the GAC filter contributed a small removal of turbidity to reach an effluent turbidity lower than the slow sand filter limit of 1 NTU. Furthermore, on Apr. 5, the GAC filter helped to produce an effluent lower than 0.3 NTU, the future proposed limit for chemically assisted filtration. Although the GAC filter occasionally contributed small removals of turbidity during runoff events, it also produced turbidity on 50% of the measurements in train 1, possibly due to GAC fines escaping the filter. It remains to be seen whether the GAC filter could be a consistent and significant contributor to turbidity removal after long-term operation and after all GAC fines are removed from the filter.

Summary of Turbidity Results from Pilot System 1

Overall, pilot system 1 (train 1) produced effluent turbidities below 1 NTU in 93% of the measurements throughout the study period, despite an average influent turbidity of 15 NTU and runoff events as high as approximately 50 NTU, periods of temperatures below 5^{o}C, and a slow sand filtration rate 0.4 m/h (operated at 0.2 m/h for a short period of time). Pilot system 1 (train 2) produced effluent turbidities below 1 NTU in 89% of the measurements, despite experiencing a period of time when the slow sand filtration rate was as high as 0.8 m/h.

The roughing filters reduced influent turbidities to an average of 10 NTU and 8.5 NTU in train 1 and 2, respectively. The average slow sand filter effluent turbidity was 0.4 NTU (standard deviation of 0.34) and 0.44 NTU (standard deviation of 0.38) in train 1 and 2, respectively, which is lower than the current standard for chemically assisted filtration.

During colder water temperatures (<5^{o}C), 100% and 86% of the effluent measurements from both trains were below 1 and 0.5 NTU, respectively. Train 1 and 2 produced an average final effluent of 0.38 NTU and 0.33 NTU, respectively, at a slow sand filtration rate of 0.4 m/h. The results from Chapter 5 were much better during this period, where approximately 99% of effluent measurements were below 0.5 NTU and the average effluent turbidity was approximately 0.27 NTU. Nevertheless, this performance was remarkable because the filters were not fully mature at the beginning of the study.

During warmer water temperatures (>10^{o}C), train 1 consistently produced a final effluent below 0.3 NTU at a slow sand filtration rate of 0.4 m/h, and train 2 consistently produced a final effluent below 0.5 NTU at a slow sand filtration rate of 0.8 m/h. These results were similar to those found in Chapter 5, however because those results were drawn from a much larger data set, the majority of the data, but not all of the data, were below the regulatory standards stated above.

Pre-ozonation did not have a significant effect on effluent turbidity, however this conclusion was drawn from a relatively small data set collected over a short period time. Pre-ozonation

may have a measurable effect on effluent turbidity over the long term, especially in warmer water temperatures.

The performance of the roughing filter and the overall system improved during warmer water conditions and benefited from increased filter maturity. However, average effluent turbidity during colder temperatures was still less than 0.5 NTU, the current Ontario effluent turbidity standard for chemically assisted filtration. Even better cold water performance is expected with a fully mature filter.

Throughout the entire study period, the roughing filter in train 1 achieved an average removal of 0.24 logs and standard deviation of 0.15, compared to an average removal of 0.29 logs and standard deviation of 0.19 in train 2. The slow sand filter in train 1 achieved an average removal of 1.25 logs and standard deviation of 0.4, compared to an average removal of 1.16 logs and standard deviation of 0.35 in train 2. The cumulative removal in train 1 was 1.49 logs with a standard deviation of 0.4, and train 2 achieved an average removal of 1.45 logs and a standard deviation of 0.39. However, it is important to recall that calculated log removals were dependent on the level in influent turbidity.

Similar to that found in Chapter 5, reducing the slow sand filter hydraulic loading rate to 0.2 m/h during runoff events proved instrumental in maintaining effluent turbidities below 1 NTU. It remains to be seen whether this would be required with a fully mature filter.

Finally, the GAC filter was not a consistent contributor to turbidity removal, although this is based on limited monitoring in relatively cold water before the filter was fully commissioned (i.e. all GAC fines removed from the filter)

6.3.1.2 Pilot System 2

Figure 6.5 shows the influent turbidity and the turbidity after each stage in pilot system 2. Figure 6.6 shows the log removal of turbidity in each stage as well as the total cumulative removal in the entire process. Over the 4-month testing period, different slow sand filtration rates were tested ranging from 0.2 to 0.6 m/h. (See Table 3.3 in Chapter 3 for the

corresponding roughing filtration rates.) It is important to recall that pilot system 2 had deeper bed depths than pilot system 1, did not use pre-ozonation, and featured a second slow sand filter in series for increased robustness. In addition, roughing filter A and roughing filter B operated in parallel, with roughing filter A feeding slow sand filter 1 from Feb. 9 to Mar. 29, and roughing filter B feeding slow sand filter 1 from Mar. 29 to May 10. The roughing filter not feeding the slow sand filter at any given time was run to waste. Both roughing filters were tested at the same hydraulic loading rates, which were generally lower than that through the roughing filters in pilot system 1.

Again, while log removals may be useful information for evaluating filter performance, it is important to note that the calculated log removals of turbidity were influenced by the level of influent turbidity (i.e. higher removals occurred during runoff events when influent turbidities were higher). Thus, it is more accurate to evaluate filter performance based on effluent turbidity levels. This is especially important for evaluating slow sand filter performance, as the turbidity of slow sand filter influent is substantially reduced by pre-roughing filtration, thus calculated log removals would be reduced.

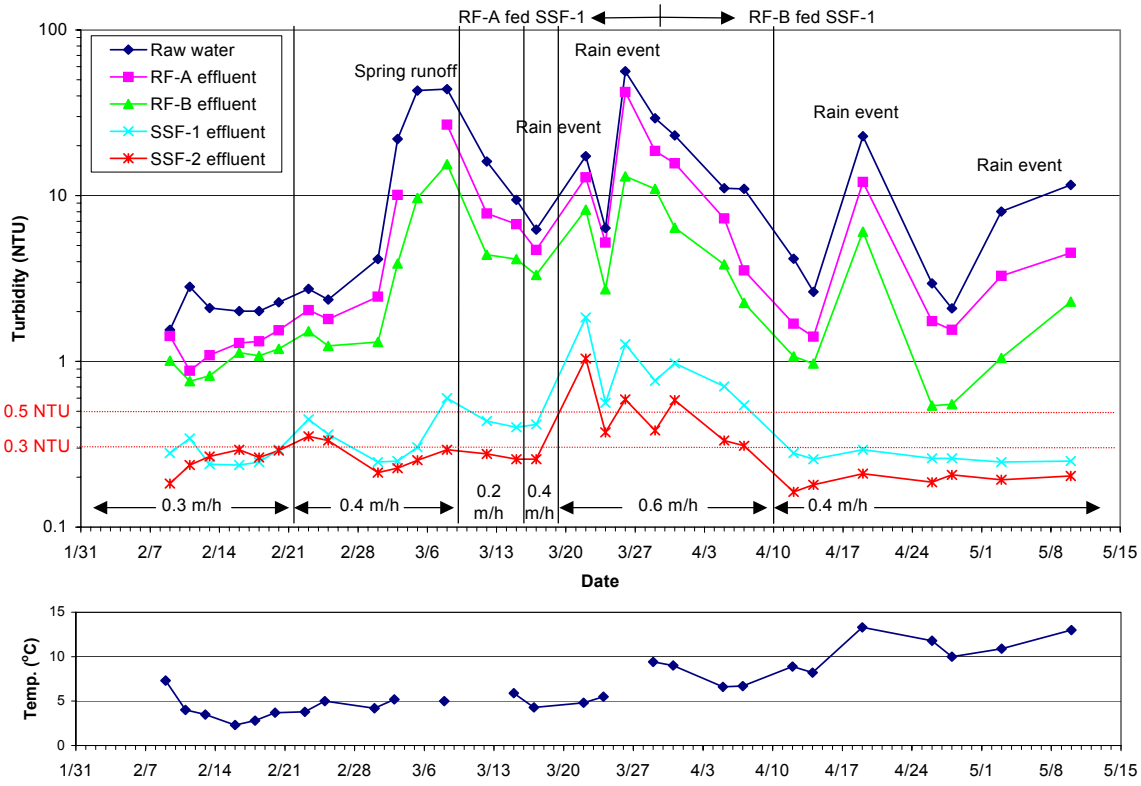

Figure 6.5: Pilot System 2 – Turbidity after each stage

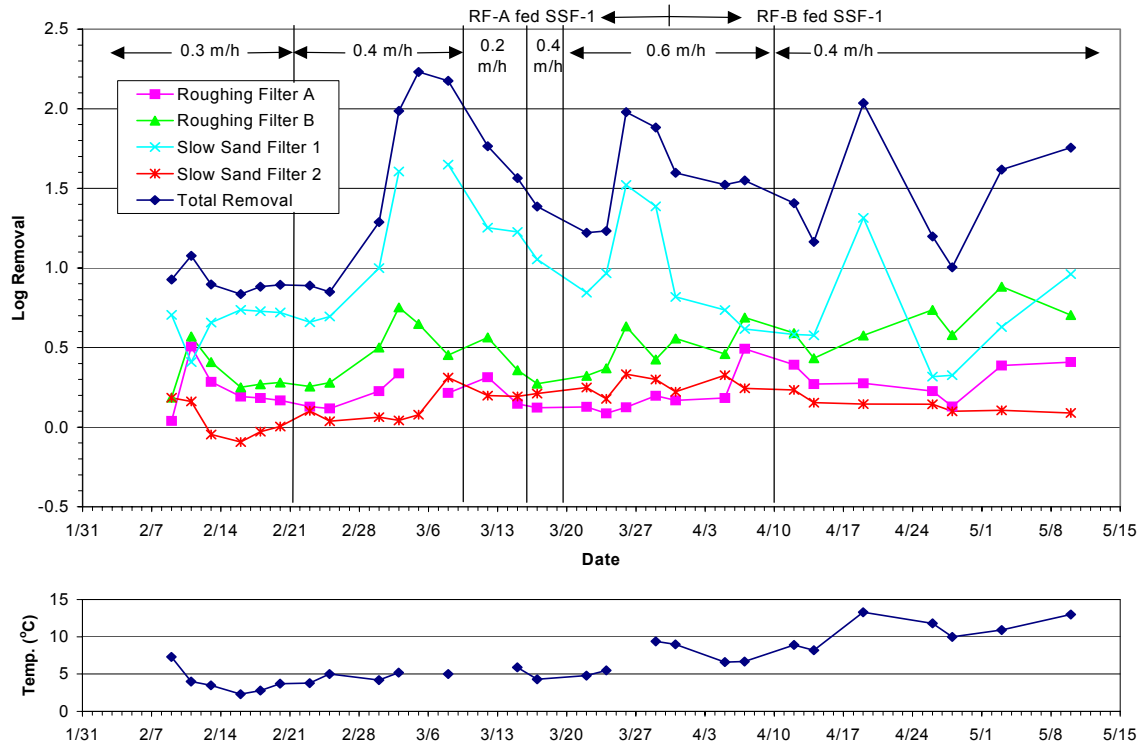

Figure 6.6: Pilot System 2 – Log removal of turbidity in each stage

Removals During Colder Water Temperatures

From Feb. 9 to Mar. 17, the slow sand filter 1 effluent was consistently below 1 NTU, despite average influent turbidities of 10.9 NTU and a spring runoff event reaching 44 NTU, temperatures below 5^{o}C, and slow sand filtration rates as high as 0.4 m/h. During this time, 93% of the slow sand filter 1 effluent measurements were below 0.5 NTU, and 53% less than or equal to 0.3 NTU. The average slow sand filter effluent turbidity was 0.34 NTU (standard deviation of 0.1) and 0.27 NTU (standard deviation of 0.04) in slow sand filter 1 and 2, respectively. This performance is remarkable for cold water conditions, especially considering that the filters were in operation for only two months prior to this study, thus were not fully biologically mature.

In the effluent of slow sand filter 2, 100% of the measurements were below 0.5 NTU, and 87% below 0.3 NTU. Clearly, slow sand filter 2 was important for meeting the effluent standard of 0.5 NTU during cold water conditions, however it remains to be seen whether it would be important with a fully mature slow sand filter 1.

The roughing filters reduced influent turbidities to an average of 5 NTU and 3.4 NTU in roughing filter A and B, respectively. Roughing filter B, with a larger bed depth of finer gravel media, consistently performed better than roughing filter A, due to its higher filtration efficiency. Removal of turbidity in the roughing filter A averaged 0.21 logs during colder water conditions, and reached a maximum of 0.5 logs. Turbidity removals were greater in roughing filter B, which averaged 0.4 logs, and reached a maximum of 0.75 logs removal.

Turbidity removals in slow sand filter 1 averaged 0.94 logs and ranged from 0.41 to 1.65 logs, compared to an average of 0.09 logs ranging from –0.09 to 0.31 logs in slow sand filter 2. Finally, cumulative turbidity removals after slow sand filter 2 averaged 1.31 logs and ranged from 0.84 to 2.23 logs. The highest removals occurred during the spring runoff when influent turbidities were higher.

Removals During Warmer Water Temperatures

From Apr. 19 to May 10, the water temperature was consistently above 10^{o}C and increased throughout the spring season. During this time, pilot system 2 consistently produced effluent below 0.3 NTU, the future proposed Canadian standard for chemically assisted filtration. This was achieved with an average influent turbidity of 9.5 NTU and runoff events with influent turbidities as high as 23 NTU, and a slow sand filtration rate of 0.4 m/h. The average slow sand filter effluent turbidity was 0.26 NTU (standard deviation of 0.02) and 0.2 NTU (standard deviation of 0.01) in slow sand filter 1 and 2, respectively. Clearly, performance was improved in warmer water temperature and a more biologically mature filter.

The roughing filters reduced influent turbidities to an average of 4.6 NTU and 2.1 NTU in roughing filter A and B, respectively. Removal of turbidity in roughing filter A averaged 0.29 logs during warmer water conditions, and reached a maximum of 0.41 logs. Turbidity removals were much greater in roughing filter B, which averaged 0.7 logs, and reached a maximum of 0.88 logs removal.

Turbidity removals in slow sand filter 1 averaged 0.71 logs and ranged from 0.32 to 1.32 logs. Further removals in slow sand filter 2 averaged 0.12 logs and ranged from 0.09 to 0.15 logs. Finally, cumulative turbidity removals after slow sand filter 2 averaged 1.52 logs and ranged from 1 to 2.04 logs.

Comparison of Performance during Cold and Warm Water Temperatures

Before results from warm and cold water temperatures can be statistically compared, it is important to determine whether the influent conditions were similar during these two time periods, as effluent turbidity levels and filter removal rates are dependent on the influent turbidity. Based on a two-tailed t-test with a 5% level of significance (see Appendix C: Table C.9 to C.10), the influent turbidity during cold water conditions (Feb. 9 to Mar. 17) was statistically similar to that during warm water conditions (Apr. 19 to May 10). Thus, the effluent results during warm and cold water conditions can be reliably compared.

Removal of turbidity in roughing filter A and B averaged 0.29 and 0.7 logs, respectively, during warm water temperatures, compared to 0.21 and 0.4 logs, respectively, during colder water temperatures. This difference was determined to be insignificant in roughing filter A, and significant in roughing filter B, based on a two-tailed t-test at a 5% level of significance (see Appendix C: Table C.9 to C.10).

In cold water conditions, the average effluent turbidity from slow sand filter 1 and 2 was 0.34 NTU and 0.27 NTU, respectively, compared to 0.26 NTU and 0.2 NTU, respectively, in warm water conditions. Based on a two-tailed t-test at a 5% level of significance, the effluent turbidity from slow sand filter 1 and 2 was significantly lower in warmer water temperatures.

Removal of turbidity in slow sand filter 1 and 2 averaged 0.71 and 0.12 logs, respectively, during warmer water temperatures, which were not statistically higher than an average of 0.94 and 0.09 logs removal, respectively, during colder water temperatures. Finally, the cumulative removal of turbidity averaged 1.52 logs in warm water, compared to 1.31 logs in cold water. Based on a two-tailed t-test at a 5% level of significance, the cumulative removal in the entire multistage system was not significantly higher than that during colder water temperatures, but it is again important to note that log removals are highly dependent on the influent turbidity, thus are not exactly indicative of system performance.

Overall, performance of pilot system 2 did improve during warmer water temperatures, but was likely due to a combination of increased biological activity in warmer temperatures and increased filter maturity.

The Effect of Increased Hydraulic Loading Rate

Pilot system 2 was challenged at a slow sand filtration rate of 0.6 m/h from Mar. 19 to Apr. 10, higher than recommended in the literature. During this time, a couple of runoff events of high raw water turbidity occurred, and the average effluent turbidity from slow sand filter 1 and 2 was 0.95 NTU (similar to that found in Chapter 5) and 0.52 NTU, respectively. On two sampling dates, the second slow sand filter was important for reducing the effluent turbidity of slow sand filter 1 to below 1 NTU. In addition, the data revealed that slow sand filter 1 and

2, operated at 0.6 m/h, consistently produced effluent less than 1 NTU and 0.5 NTU, respectively, only when raw water turbidity was less than approximately 10 to 15 NTU (these results were similar to those found in Chapter 5). It remains to be seen, however, if slow sand filter 2 would be as important with a fully mature slow sand filter 1.

Nevertheless, it is generally not recommended in normal practice to operate slow sand filters at such high hydraulic loading rates during runoff events of high influent turbidity.

The Significance of a Second Slow Sand Filter in Series

In addition to its effectiveness during increased hydraulic loading rates, slow sand filter 2 consistently produced effluent turbidity lower than slow sand filter 1 throughout the entire study period, with the exception of 3 sampling dates in February. Overall, the average effluent turbidity from slow sand filter 1 was 0.47 NTU (standard deviation of 0.36), compared to an average effluent turbidity of 0.31 NTU in slow sand filter 2 (standard deviation of 0.17). Based on a two-tailed t-test with a 5% level of significance, the effluent turbidity from slow sand filter 2 was significantly less than that of slow sand filter 1.

Practically, the second sand filter may only be required during runoff events, periods of increased hydraulic loading, and cleaning of the first sand filter. In full scale plants, the second sand filter would likely have a reduced filter area and operate at a higher hydraulic loading rate, as there would be minimal headloss development across the filter (due to its minimal solids loading).

The Effect of Roughing Filter Design on the Removal of Turbidity

It is important to determine if the different roughing filter designs used in pilot system 1 and 2 had a significant impact on filter performance in terms of effluent turbidity and log removal of turbidity. In this section, roughing filter A, a traditional design based on recommendations in the literature, is first compared to roughing filter B, which contained deeper bed depths of finer gravel layers. In addition, roughing filter A from pilot system 2 is compared to the roughing filters of both trains in pilot system 1, which had half the total bed depth of the

roughing filters in pilot system 2, albeit finer media fractions. For more details on the above mentioned roughing filter design parameters, refer to Chapter 3.

Roughing filter B consistently had lower effluent turbidity and higher removal of turbidity than roughing filter A during the entire study. Based on a two-tailed paired t-test with a 5% level of significance, as expected this difference in performance was significant (see Appendix C: Table C.12 to C.13).

A similar paired t-test was conducted to determine if roughing filter A in pilot system 2 performed better than the roughing filters in pilot system 1 (in both train 1 and train 2). Results of the paired t-test indicated that there was no significant difference in performance in terms of effluent turbidity and log removal of turbidity (see Appendix C: Table C.14). However, since roughing filter B performed significantly better than roughing filter A in pilot system 2, it is assumed that roughing filter B performed significantly better than both roughing filters in pilot system 1.

Thus, the design of roughing filter B, which had a higher filtration efficiency, had a significant impact on roughing filter performance, when compared to roughing filter A in pilot system 2 and both roughing filters in pilot system 1. Roughing filter B, however, experienced some operational problems in warmer water temperatures during June, when the headloss across the filter increased to greater than 20 cm, likely due to excessive growth of biomass in the finer media. During this time, cleaning of the roughing filter by gravity flushing (rapid drainage of the filter to flush away solids accumulations) was hindered by lowered drainage velocities. However, the headloss returned to 5 cm and drainage velocities returned to normal in July, possibly due to a re-establishment of equilibrium in the biomass population in the warmer water conditions. It remains to be seen whether the operability of roughing filter B will become a problem, especially during prolonged periods of warmer water temperatures, when increased biological growth and algal build-up in the filter may dramatically impede drainage velocities during cleaning.

Summary of Turbidity Results from Pilot System 2

Overall, pilot system 2 produced effluent turbidity below 1 NTU in 97% of the measurements throughout the study period, despite an average influent turbidity of 12.8 NTU and runoff events as high as 56 NTU, periods of temperatures below 5^{o}C, and a period of hydraulic loading as high as 0.6 m/h in the slow sand filter. Effluent turbidity was below 0.5 NTU in 90% of the measurements, and below 0.3 NTU in 69% of the measurements. The average effluent turbidity in slow sand filter 1 and 2 was 0.47 NTU (standard deviation of 0.36) and 0.31 NTU (standard deviation of 0.17), respectively. Slow sand filter 2 produced significantly lower effluent turbidities than slow sand filter 1. Slow sand filter 1 achieved an average removal of 0.88 logs and standard deviation of 0.37, compared to an average removal of 0.15 logs and standard deviation of 0.11 in slow sand filter 2. The cumulative removal was 1.41 logs with a standard deviation of 0.44.

Similar to that found in Chapter 5, the second slow sand filter was also important for reducing effluent turbidity to below 1 NTU during runoff events of high turbidity combined with increased hydraulic loadings through slow sand filter 1. In fact, at a hydraulic loading of 0.6 m/h, slow sand filter 2 reduced the average slow sand filter 1 effluent from 0.95 to an average of 0.52 NTU. It was also important for meeting the standard of 0.5 NTU during cold water conditions. However, it remains to be seen whether it would important with a fully mature slow sand filter 1.

During cold water temperatures (<5^{o}C), slow sand filter 1 consistently produced an effluent below 1 NTU and 93% of the measurements were below 0.5 NTU, with a slow sand filtration rate of 0.4 m/h. The average effluent turbidity of slow sand filter 1 and 2 was 0.34 and 0.27 NTU, respectively. These results were somewhat better than those found in Chapter 5, where 94% and 82% of effluent measurements were less than 1 NTU and 0.5 NTU, respectively. Nevertheless, this performance is remarkable considering that the system was not fully mature.

During warm water temperatures (>10^{o}C) and with a more mature filter, pilot system 2 consistently produced a final effluent below 0.3 NTU, thus it benefited from increased

maturity and increased biological activity in warmer temperatures. These results were similar to that found in Chapter 5, however because those results were drawn from a much larger data set, the majority of the data, but not all of the data, were below the regulatory standard stated above.

Effluent turbidity from slow sand filter 1 and 2 was significantly lower in warmer water temperatures. However, the log removal of turbidity in slow sand filter 1 and 2, and cumulative removal throughout the entire system was not significantly improved during warmer water conditions. However, it is important to recall that log removals are typically dependent on the level of influent turbidity, and may not be truly indicative of filter performance.

Roughing filter A and B reduced influent turbidities to an average of 7.2 NTU and 3.8 NTU, respectively. Throughout the entire study period, roughing filter A achieved an average removal of 0.23 logs and standard deviation of 0.12, compared to an average removal of 0.48 logs and standard deviation of 0.18 in roughing filter B. Effluent turbidity and log removal of turbidity in roughing filter B was significantly better than that of roughing filter A, and both roughing filters in pilot system 1, most likely due to its deeper bed depth of finer gravel media. In addition, removal of turbidity in roughing filter B was significantly greater in warm water conditions, whereas water temperature did not have a significant effect on removals in roughing filter A. However, due to its larger headloss and potential filter cleaning problems, its operational integrity remains in question, and should be monitored over a long-term period of warmer water temperatures.

6.3.2 Total and Fecal Coliforms

The following section analyzes and discusses the removal of total and fecal coliforms in each stage of the multistage filter, as well as the cumulative removal in the entire process. The raw data used to generate the following graphs can be found in Appendix D (Table D.1 to D.2). Hydraulic loading conditions are shown on the graphs, and the influent turbidity and water temperatures are shown below each graph.

The total coliform method had a detection limit of <2 MPN/100 mL. For results that were <2 MPN/100 mL, log removals were calculated by using 2 MPN/100 mL. This resulted in calculated log removals that were conservative. The resulting log removals were reported as "greater than" the calculated removal. On the graphed results, <2 MPN/100 mL was represented by showing 1 MPN/100 mL as a "dummy" number.

The fecal coliform method had a detection limit of 0 CFU/100 mL. However, to calculate log removals for results of 0 CFU/100 mL, the number 0.1 CFU/100 mL was used. The resulting log removals were therefore understated, and were reported as '"greater than" the calculated removal. Due to the log scale on the graphed results, 0 CFU/100 mL was shown as 0.1 CFU/100 mL.

Again, it is important to recall that calculated log removals are typically dependent on influent levels (ie. higher log removals typically result with higher influent levels).

6.3.2.1 Pilot System 1 (Train 1)

Figure 6.7 and Figure 6.8 show the total and fecal coliforms after each stage in pilot system 1 (train 1), respectively. Figure 6.9 shows the log removal of total coliforms in each stage as well as the total cumulative removal. The data presented in this section is discussed in chronological order of when the sampling events took place.

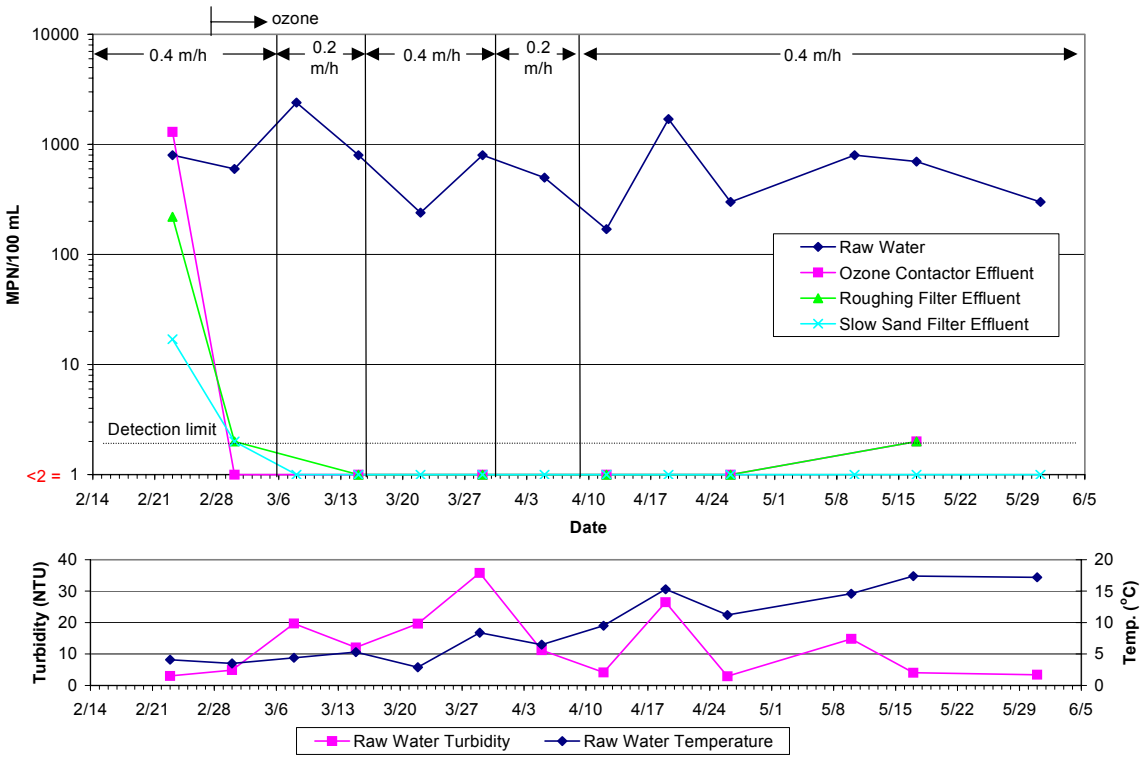

Figure 6.7: Pilot System 1 (Train 1) – Total coliforms after each stage

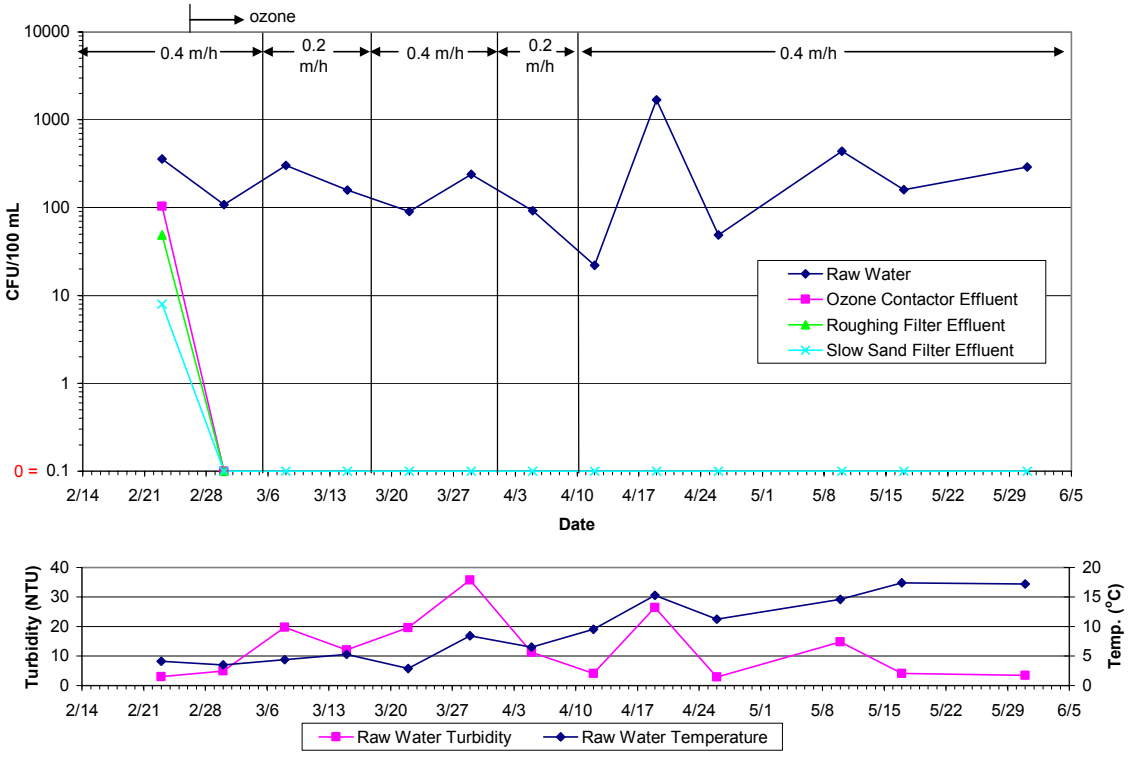

Figure 6.8: Pilot System 1 (Train 1) – Fecal coliforms after each stage

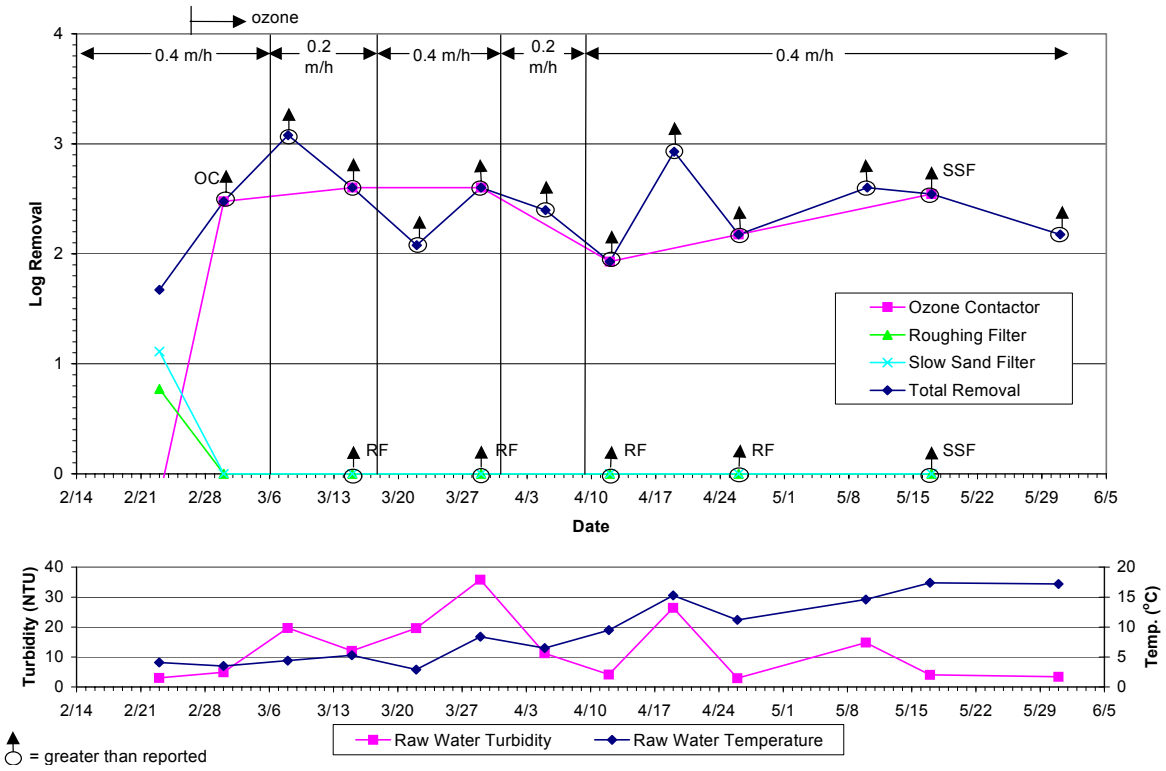

Figure 6.9: Pilot System 1 (Train 1) – Log removal of total coliforms in each stage

Removal of coliforms in both pilot systems was uncharacteristically poor on Feb. 23, compared to the rest of the data set. In pilot system 1 (train 1), with the temperature at 4.1°C, total coliforms were reduced from 800 MPN/100 mL in the raw water to 220 MPN/100 mL in the roughing filter effluent to 17 MPN/100 mL in the slow sand filter effluent. This translates to a log removal of 0.6 in the roughing filter, 1.1 in the slow sand filter, and a total of 1.7 logs. Similarly, fecal coliforms were reduced from 360 CFU/100 mL in the raw water to 49 CFU/100 mL in the roughing filter effluent to 8 CFU/100 mL in the slow sand filter effluent. The incomplete removal of coliforms in this initial sampling could be due to a combination of reduced biological activity in the slow sand filter at lower water temperatures and inadequate biological maturity of the filter prior to the beginning of this study. However, performance was much better on other days (in Feb. and Mar.) with similar operating conditions, as is seen in the results from train 2 in the next section. Thus, other causes were suspected, such as coliforms associated with negatively charged colloidal matter, which is generally difficult to

remove in passive filters. In normal practice, slow sand filtration would usually be followed by post-chlorination to protect against pathogen breakthrough in an occurrence such as this.

After Feb. 23, however, pre-ozonation was introduced to the process and the total coliforms in the ozone contactor effluent were consistently ≤2 MPN/100 mL for the rest of the study period, despite temperatures ranging below 5 ^{o}C, and variable raw water levels ranging from 170 to 2400 MPN/100 mL. During this time, cumulative removal of total coliforms ranged from >1.9 to >3.1 logs after pre-ozonation. On all days that the ozone contactor was sampled, all removal of total coliforms occurred in the ozone contactor. At an estimated aqueous dose of 4 mg/L and contact time of approximately 10 minutes, these results were expected.

Similarly, after pre-ozonation, the ozone contactor effluent on Mar. 1 had 0 CFU/100 mL. Unfortunately, fecal coliforms in the ozone contactor effluent were not sampled after this date. However, the slow sand filter effluent consistently had 0 CFU/100 mL for the rest of the study period. Since all removal of total coliforms occurred in the ozone contactor, it is assumed that removal of fecal coliforms also occurred in the ozone contactor.

Overall, although the main purpose of pre-ozonation was for removal of dissolved organic carbon and colour, it achieved excellent removal of total coliforms to below detection limits and complete removal of fecal coliforms.

6.3.2.2 Pilot System 1 (Train 2)

Figure 6.10 and Figure 6.11 show the total and fecal coliforms after each stage in pilot system 1 (train 2), respectively. Figure 6.12 shows the log removal of total coliforms in each stage as well as the total cumulative removal.

Removal of coliforms in the individual stages of the multistage filter was explored by looking at the results from train 2, as pre-ozonation did not commence in this train until Mar. 22. Until this date, there was no inactivation of coliforms in the ozone contactor, thus allowing the rest of the system to provide treatment.

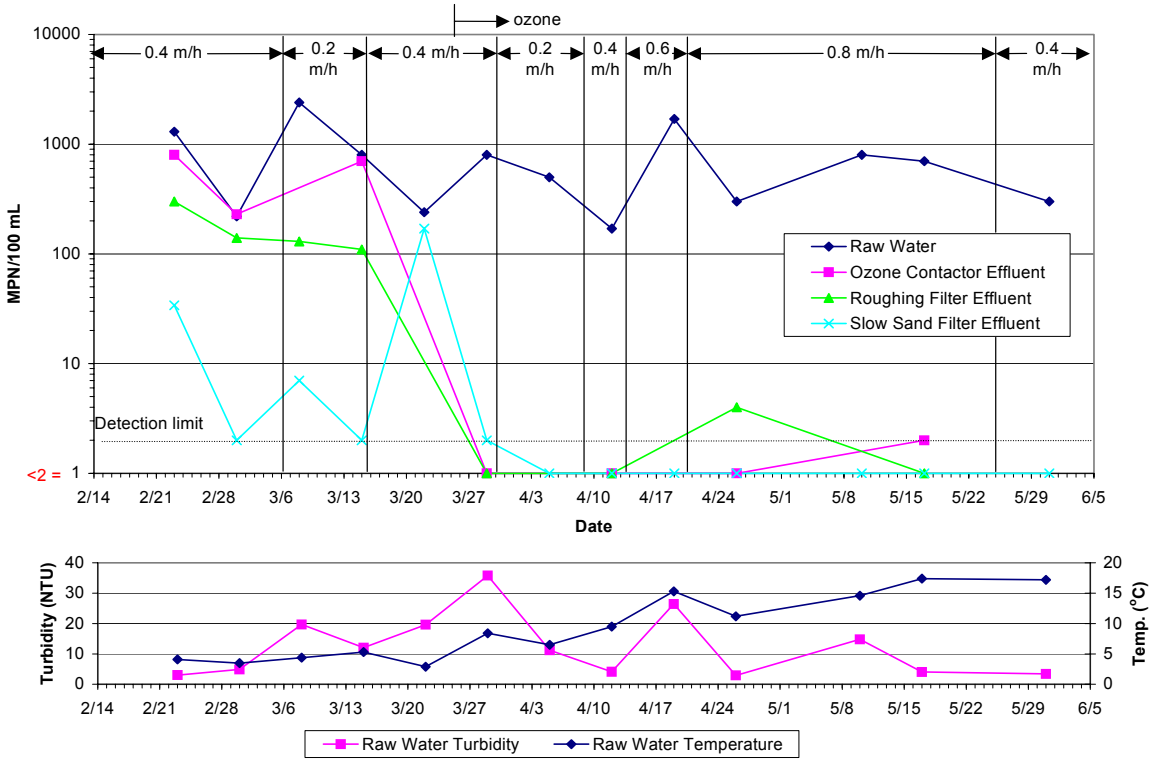

Figure 6.10: Pilot System 1 (Train 2): Total coliforms after each stage

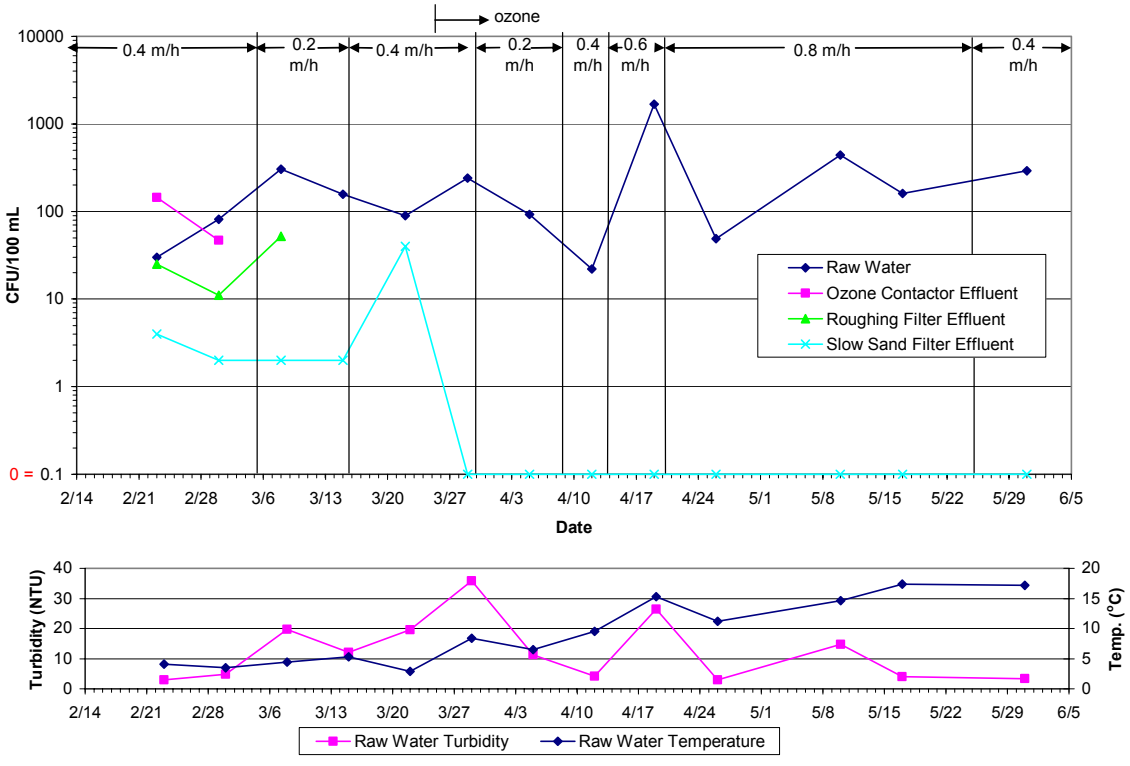

Figure 6.11: Pilot System 1 (Train 2): Fecal coliforms after each stage

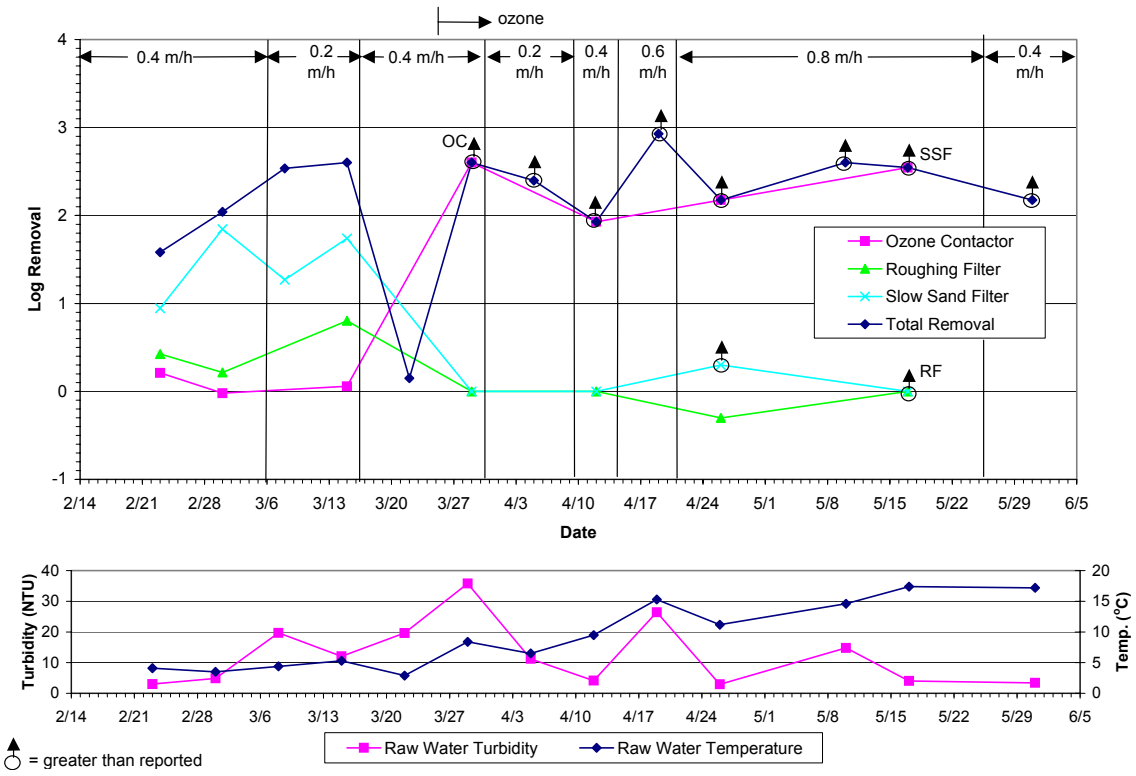

Figure 6.12: Pilot System 1 (Train 2): Log removal of total coliforms in each stage

Removals during Colder Water Temperatures and without Pre-Ozonation

Before Mar. 22, raw water total coliforms ranged from 220 to 2400 MPN/100 mL and the water temperature ranged from 3 to 5°C. The corresponding roughing filter effluent concentrations were 140 and 130 MPN/100 mL. This translated to a log removal ranging from 0.2 to 1.3 in the roughing filter. The fecal coliforms in the raw water ranged from 30 to 304 CFU/100 mL, and roughing filter effluent ranged from 11 to 52 CFU/100 mL. Hence, without pre-ozonation, the roughing filter was a substantial contributor to the removal of coliforms.

Furthermore, total coliforms in the slow sand filter effluent ranged from 2 to 7 MPN/100 mL, with the exception of 34 MPN/100 mL on Feb. 23, when all pilot systems exhibited poor removals. Fecal coliforms in the slow sand filter effluent ranged from 2 to 4 CFU/100 mL. Log removals of total and fecal coliforms in the slow sand filter ranged from 0.95 to 1.9 logs, and 0.74 to 1.42 logs, respectively. Finally, the total cumulative removal of total coliforms in

the multistage filter ranged from 1.6 to 2.6 logs, with 3 out of 4 measurements above 2 logs removal. Total cumulative removal of fecal coliforms ranged from 0.88 to 2.2 logs, with 3 out of 4 measurements above 1.6 logs removal. Although removals greater than 2 logs have been reported in the slow sand filtration literature (e.g. Van Dijk and Ooman (1978)), it is important to remember that log removals are greatly influenced by the level of influent (i.e. higher log removals with higher influent levels).

Overall, the roughing filter was a significant contributor to coliform removal during cold water conditions. However, the multistage filter system, without pre-ozonation, did not achieve complete removals of coliforms on all sampling dates. This was likely due to inadequate biological maturity of the filter prior to the beginning of the study combined with reduced biological treatment in the colder water temperatures, as well as a reduced bed depth (~0.5 m) in the slow sand filter. It is expected that performance in cold temperatures would be improved with a mature filter, however further testing is required to confirm this.

Event of Poor Performance

During the runoff event on Mar. 22, the slow sand filter effluent had uncharacteristically high total and fecal coliform concentrations of 170 MPN/100 mL and 40 CFU/100 mL, respectively. Although influent turbidities were around 20 NTU, the influent total and fecal coliforms were only 240 MPN/100 mL and 90 CFU/100 mL. Overall, the total cumulative removal of total and fecal coliforms in the multistage filtration system was only 0.15 and 0.35 logs, respectively, which was much lower than expected. The cause of this poor performance is uncertain, although it is important to note that all pilot systems exhibited poor performance on this date. It is suspected that a high proportion of colloidal matter could have been present in the influent, as removal of turbidity was also poor on this date. Nevertheless, in North American practice, slow sand filtration would normally be followed by chlorination, which would have protected against pathogen breakthrough in a potentially relatively rare occurrence such as this.

Removals with Pre-Ozonation

After Mar. 22, pre-ozonation was introduced to the process and total coliforms in the ozone contactor effluent were consistently ≤2 MPN/100 mL despite variable raw water levels ranging from 170 to 1700 MPN/100 mL. Cumulative removal of total coliforms ranged from >1.9 to >2.9 logs after ozonation was introduced. On all days that the ozone contactor was sampled, all removal occurred in the ozone contactor. Fecal coliforms in the ozone contactor effluent were not sampled during this time. However, the slow sand filter effluent consistently had 0 CFU/100 mL for the duration of the pre-ozonation period. Since all removal of total coliforms occurred in the ozone contactor, it is assumed that removal of fecal coliforms also occurred in the ozone contactor.

A comparison of the cumulative removal of total and fecal coliforms between both trains is shown in Figure 6.13 and Figure 6.14, respectively. In particular, the effect of pre-ozonation can be seen from Feb. 23 to Mar. 22. During this time period, cumulative removal of total and fecal coliforms was consistently higher in train 1 (with pre-ozonation) than in train 2 (without pre-ozonation). After Mar. 22, both trains were operating with pre-ozonation and log removals of total and fecal coliforms were identical between both trains.

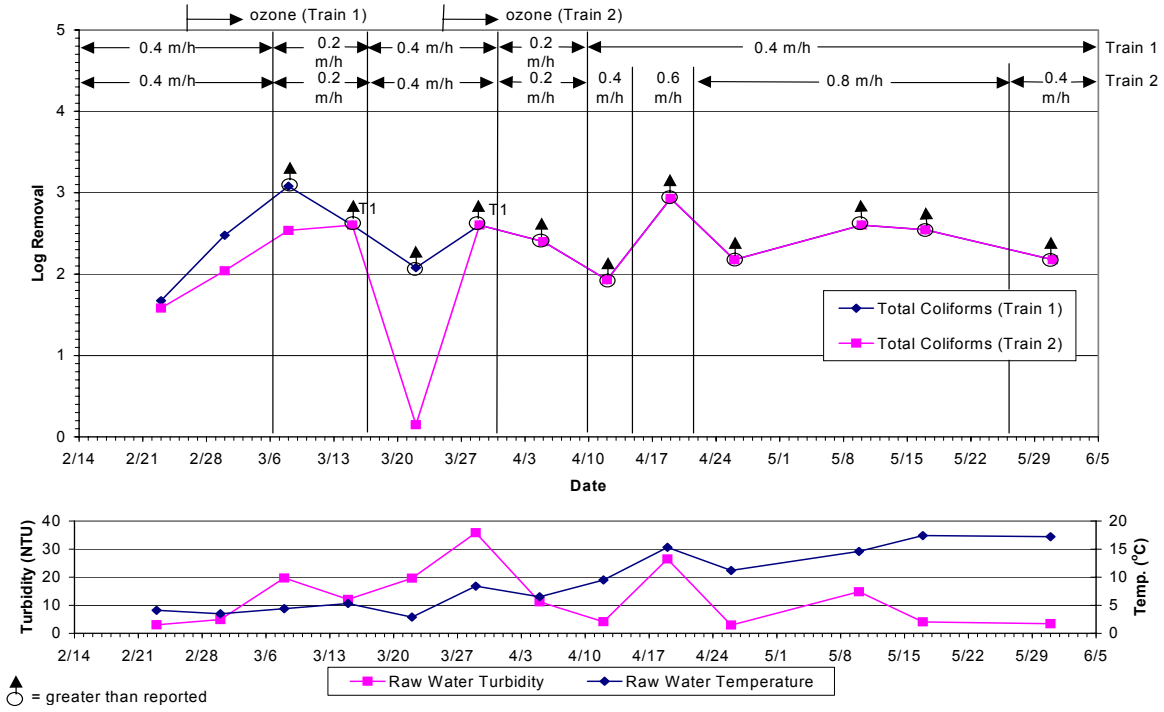

Figure 6.13: Pilot System 1: Cumulative removal of total coliforms in train 1 and train 2

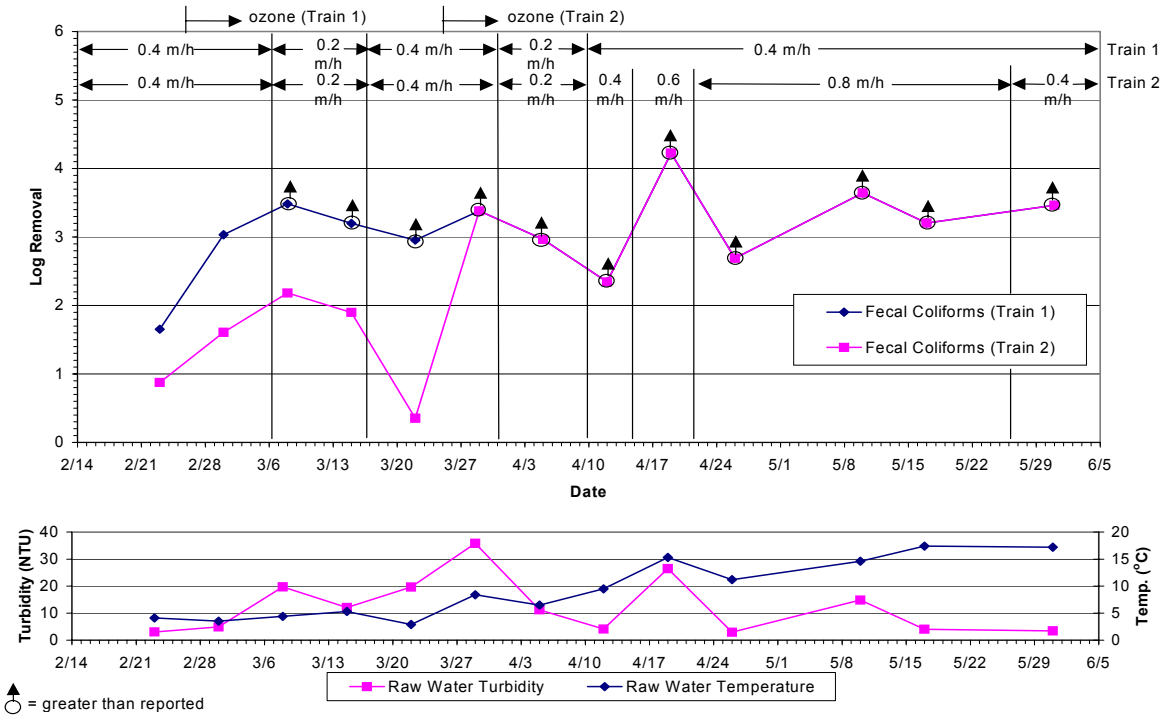

Figure 6.14: Pilot System 1: Cumulative removal of fecal coliforms in train 1 and train 2

Summary of Total and Fecal Coliform Results from Pilot System 1 (Train 2)

Although pre-ozonation in pilot system 1 was mainly used for removal of dissolved organic carbon and colour, it consistently achieved complete removal of total and fecal coliforms before the filtration stages, including during cold water conditions. This result was not unexpected with the estimated aqueous dose of 4 mg/L. Without pre-ozonation, the total coliforms in train 2 effluent was ≤2 MPN/100 mL in only 2 out of 5 of the measurements. It is important to note, however, that the roughing filter and slow sand filter had shallower bed depths than recommended in the literature, and improved filtration would be expected with deeper bed depths. In addition, this period presented the coldest water temperatures and improved removal would be expected with a fully mature filter, however this remains to be seen. Furthermore, it remains to be seen whether pilot system 1 could achieve complete removal of coliforms in warmer water conditions without the use of pre-ozonation.

6.3.2.3 Pilot System 2

Figure 6.15 and Figure 6.16 show the total and fecal coliforms after each stage in pilot system 2. Figure 6.17 shows the log removal of total coliforms in each stage. Data were collected for various hydraulic loading rates including 0.4 m/h, 0.2 m/h during runoff events, and 0.6 m/h. It is recalled that pilot system 2 did not use pre-ozonation and had deeper bed depths in the roughing filter and slow sand filter than in pilot system 1. In addition, roughing filter A and roughing filter B operated in parallel, with roughing filter A feeding slow sand filter 1 from Feb. 9 to Mar. 29, and roughing filter B feeding slow sand filter 1 from Mar. 29 to May 10.

Removals during Colder Water Temperatures

Before Mar. 22, influent total coliform levels ranging from 170 to 2400 MPN/100 mL were reduced to ≤2 MPN/100 mL in 3 out of 4 measurements, despite temperatures below 5^{o}C. These results were better than that from pilot system 1 during cold temperatures, likely due to the deeper bed depths.

Removals in roughing filter A ranged from 0 to 0.9 logs, whereas roughing filter B achieved 0.4 to 1.2 logs, likely due to its increased filtration efficiency. Removals in slow sand filter 1

ranged from 1.7 to 2.2 logs, with the exception of 0.6 logs on Feb. 23, when all pilot systems exhibited poor performance. Interestingly, during the event of poor performance on Feb. 23, the second slow sand filter was instrumental in reducing total coliform levels from 130 to 30 MPN/100 mL, adding an additional 0.6 logs removal to achieve a cumulative removal of 1.4 logs in the multistage system. Overall, the cumulative removals of total coliforms in the multistage system ranged from 1.4 to 3.1 logs.

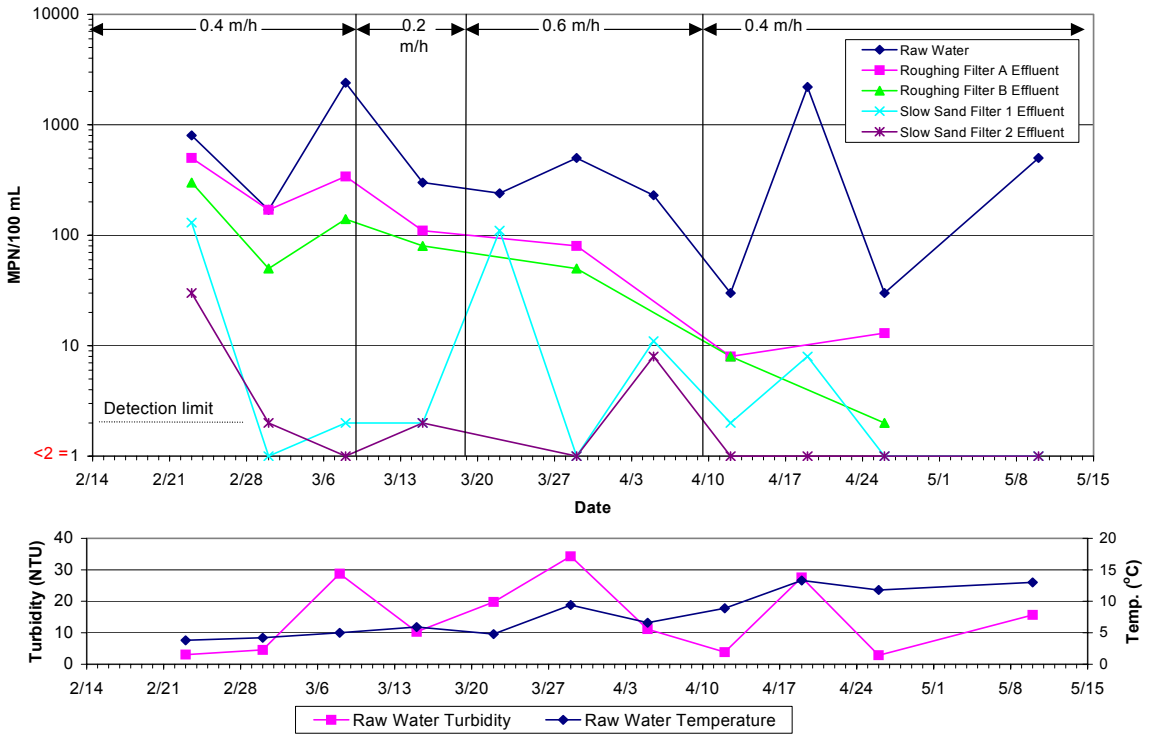

Figure 6.15: Pilot system 2 – Total coliforms after each stage

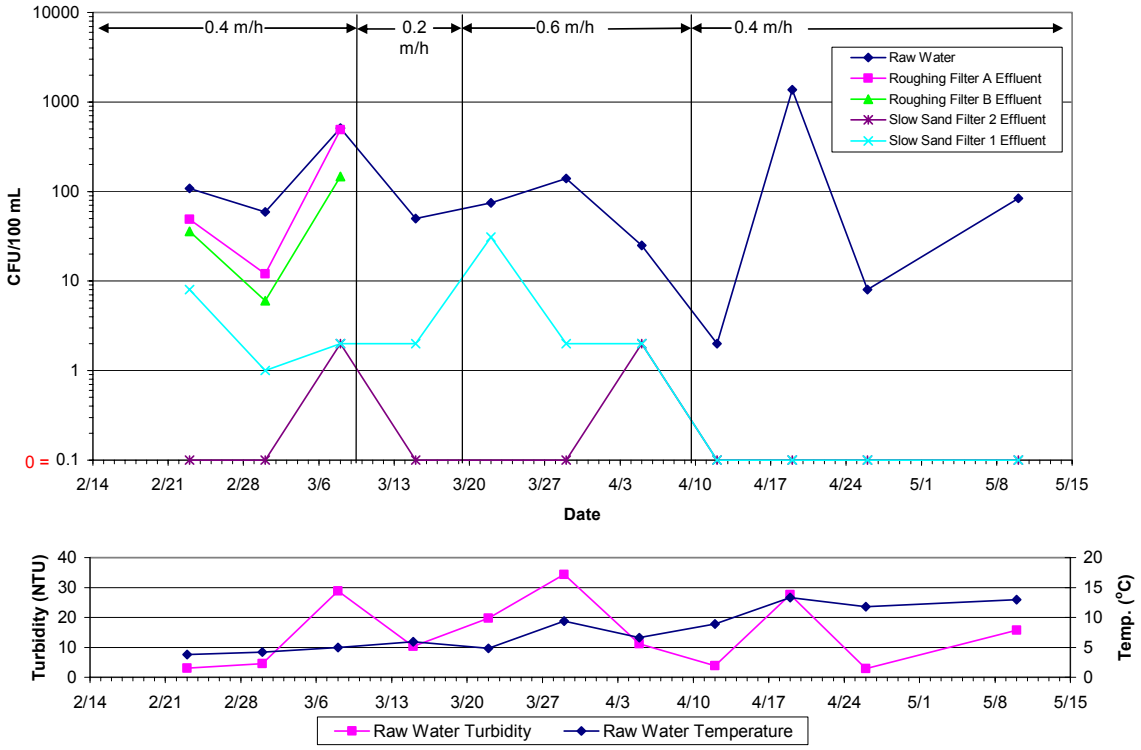

Figure 6.16: Pilot system 2 – Fecal coliforms after each stage

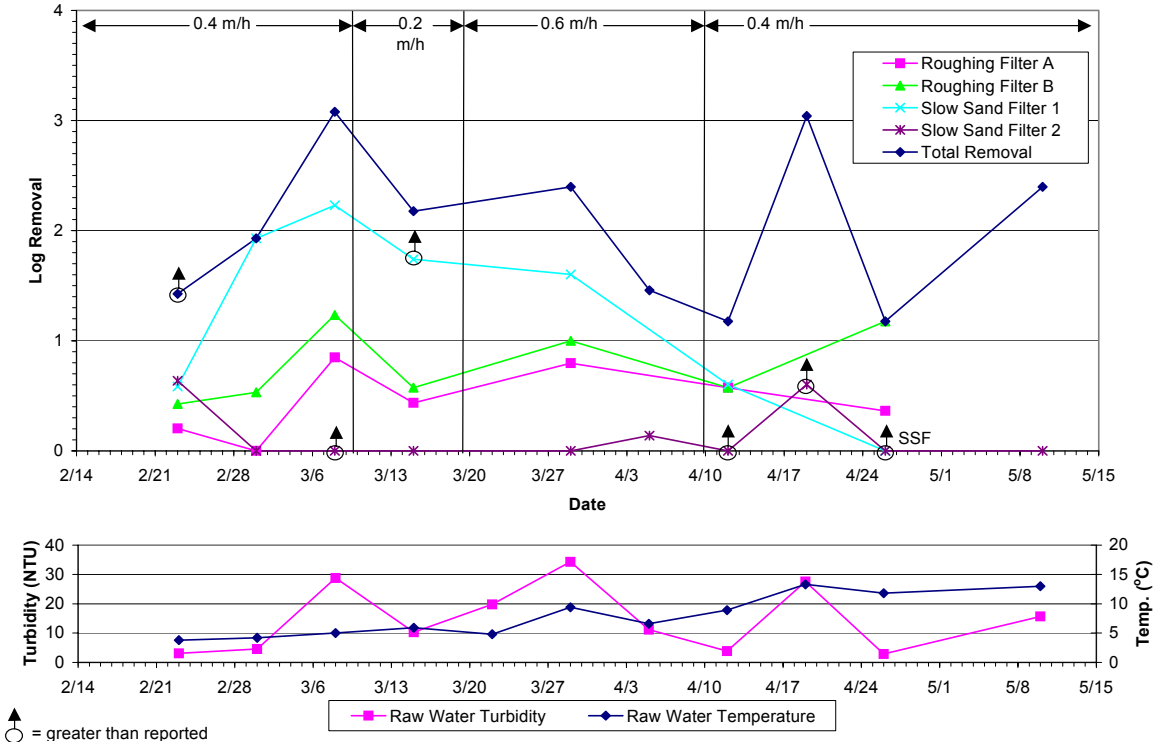

Figure 6.17: Pilot system 2 – Log removal of total coliforms in each stage

During the same time period, influent fecal coliform levels ranging from 50 to 510 CFU/100 mL were reduced to 0 CFU/100 mL in 3 out of 4 measurements, despite temperatures below 5^{o}C. Removals in roughing filter A ranged from 0 to 0.7 logs, compared to 0.5 to 1 logs in roughing filter B. Removals in slow sand filter 1 ranged from 1.1 to 2.4 logs, with the exception of 0.8 logs on Feb. 23, when all pilot systems exhibited poor performance. Slow sand filter 2 contributed removals on 3 out of 4 measurements, which ranged from 1 to 1.3 logs, and as high as 1.9 logs to achieve complete removal on Feb. 23, when slow sand filter 1 exhibited reduced performance with an effluent of 8 MPN/100mL. The cumulative removals of fecal coliforms for the multistage system ranged from 2.4 to 3, which was remarkable considering the cold water temperatures.

Similar to pilot system 1, pilot system 2 also exhibited poor performance on Mar. 22, which resulted in a cumulative 0.34 log removal of total coliforms through the roughing filter and slow sand filter 1. Unfortunately, the effluent from slow sand filter 2 was not sampled on this date, which would have determined its impact in an occurrence such as this. It is important to note that although the hydraulic loading rate through slow sand filter 1 was 0.6 m/h on this date, pilot system 1 also experienced compromised performance at 0.4 m/h, thus the increased hydraulic loading was not suspected as a contributor to poor performance on this date. Although, had the hydraulic loading rate been lower, the system may have performed better.

Overall, with the exception of two sampling dates, removal of total and fecal coliforms was excellent considering that the filter was likely not fully mature at the beginning of the study, and despite the ambient water temperatures below 5^{o}C. The deeper bed depths were suspected to have contributed to improved removal, when compared to results from pilot 1. In addition, roughing filter B consistently outperformed roughing filter A with respect to effluent coliform levels and log removal of coliforms, likely due to its increased filtration efficiency.

Removals during Increased Hydraulic Loading Rates

On Mar. 29 and Apr. 5, the performance of pilot system 2 at an increased hydraulic loading rate of 0.6 m/h (through the slow sand filter) was further investigated. On Mar. 29, total coliforms were reduced from 500 to <2 MPN/100 mL. Removal of total coliforms in roughing filter A was 0.8 logs, compared to 1 log in roughing filter B. Slow sand filter 1 added an extra 1.6 logs removal to obtain complete removal of total coliforms, resulting in a cumulative removal of 2.4 logs in the entire multistage filter. Thus, the increased hydraulic loading did not have a detrimental effect of effluent quality on this sampling date.

On Apr. 5, total coliforms were reduced from 230 MPN/100 mL to 11 MPN/100 mL through the roughing filter and slow sand filter 1, a cumulative removal of 1.3 logs. In this instance, removal was incomplete, and slow sand filter 2 further reduced levels to 8 MPN/100 mL, an additional removal of 0.1 logs, for a cumulative removal of 1.5 logs in the entire multistage filtration system. The incomplete removal of coliforms on this date could only be attributed to a combination of the potential presence of colloidal matter in the influent and the increased hydraulic loading rate of 0.6 m/h through the slow sand filter.

Thus, as performance was inconsistent during increased hydraulic loadings, the effect of increased hydraulic loading rate on the removal of coliform bacteria could not be confirmed.

Removals during Warmer Water Temperatures

After April 9, the water temperatures were warmer (above 9^{o}C) and the hydraulic loading rate through slow sand filter 1 was 0.4 m/h. Even with influent total and fecal coliform levels ranging from 30 to 2200 MPN/100 mL and 2 to 1377 CFU/100 mL, respectively, the multistage filter system consistently reduced total coliforms to <2 MPN/100 mL and fecal coliforms to 0 CFU/100 mL. In addition, roughing filter B continued to perform better than roughing filter A. In particular, on April 26, removal of total coliforms was 1.2 logs in roughing filter B, compared to only 0.4 logs in roughing filter A.

The second slow sand filter was again important during a runoff event on April 19, when influent total coliform levels were 2200 MPN/100 mL and it reduced slow sand filter 1

effluent from 8 to <2 MPN/100 mL, an additional removal of 0.6 logs, resulting in a cumulative removal of >3 logs.

The Impact of a Second Slow Sand Filter During Cold Water Temperatures

In Figure 6.18, the cumulative removal of fecal coliforms after each stage in pilot system 2 is shown for the entire study period. It is evident that during the warmer water temperatures, most removal of fecal coliforms was achieved in slow sand filter 1. It is important to note that in Figure 6.18, from Apr. 5 to May 10, the line indicating the cumulative removal after slow sand filter 2 is overlapped by the line indicating the cumulative removal after slow sand filter 1, thus there was no additional removal occurring in slow sand filter 2 during this time. During the colder water temperatures however, when biological treatment was hindered and the system was not as mature, slow sand filter 2 contributed additional treatment up to 1.9 logs removal. Thus, slow sand filter 2 was important for providing additional treatment during colder water temperatures. Its importance at low temperatures in a fully mature system remains to be determined.

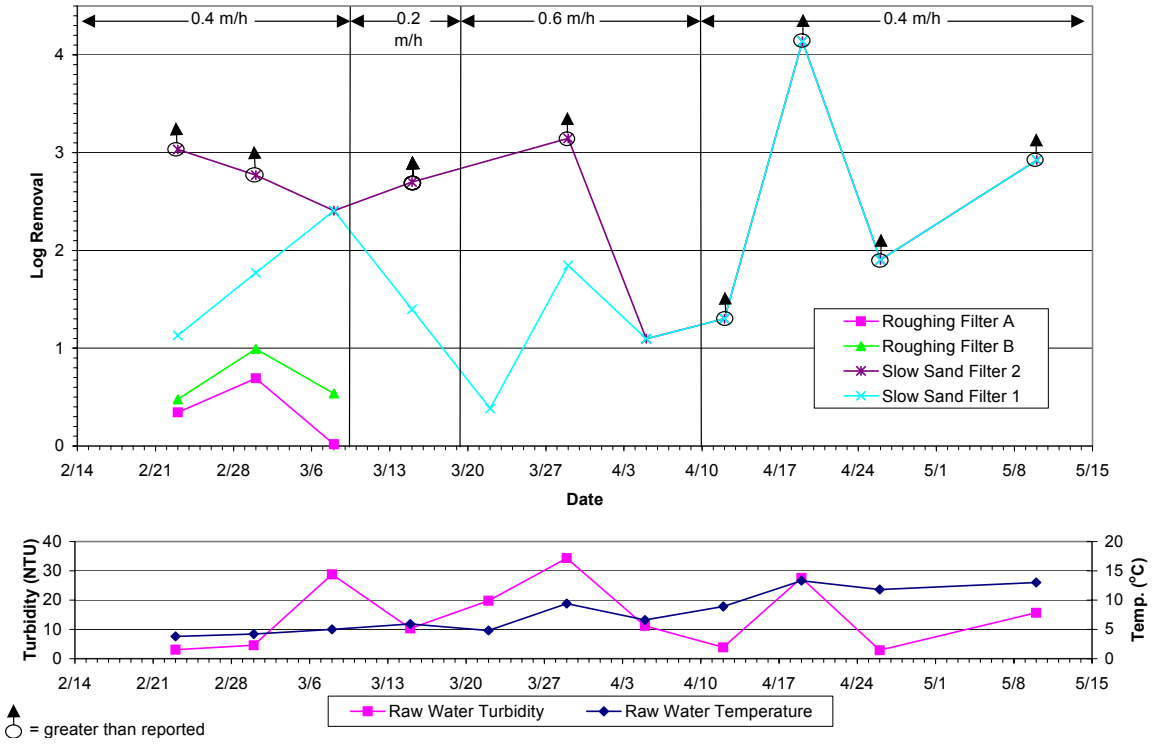

Figure 6.18: Pilot system 2 – Cumulative removal of fecal coliforms after each stage

Summary of Total and Fecal Coliform Results from Pilot System 2

Overall, pilot system 2, which operated without pre-ozonation, reduced total coliforms to ≤2 MPN/100 mL and fecal coliforms to 0 CFU/100 mL in 80% of the measurements, despite influent levels as high as 2400 MPN/100 mL and 1377 CFU/100 mL, periods of water temperature less than 5°C, and a period of hydraulic loading rate at 0.6 m/h.

Removal was complete in 3 out of 4 measurements during cold water conditions (<5°C) and 100% of measurements during warm water conditions. The deeper bed depths in the roughing filter and slow sand filter likely contributed to improved performance during colder temperatures with a biologically immature filter. Again, improved performance during colder temperatures would be expected with a fully mature filter, however this remains to be determined.

Roughing filter B consistently outperformed roughing filter A, likely due to its increased filtration efficiency from deeper bed depths of finer gravel media. However, due to its larger headloss and potential filter cleaning problems discussed in previous sections, its operational integrity remains in question, and should be monitored over a long-term period of warmer water temperatures.

Finally, the second slow sand filter was, on a number of occurrences, important for providing additional treatment during periods of higher influent levels, inadequate performance in the previous stages, increased hydraulic loadings, and colder water temperatures. Its importance subsequent to treatment in a fully mature roughing and slow sand filter remains to be determined.

6.3.3 *Cryptosporidium* Challenge Test Results

The results for the first set of *Cryptosporidium* challenge tests, namely Test 5a, 5b, and 5c, as described in Chapter 3, are shown in Table 6.1, Table 6.2, and Table 6.3, respectively. It is important to note that the challenge tests were conducted on the slow sand filter only, not the entire system. In each table, the sample number is represented by the sample location,

followed by the sampling time. For example, sample '2*-3' was sampled from sampling port # 2* (see Chapter 3 for details on sampling port locations) at the third hour of the challenge test. Count 1 and Count 2 represent microscopic enumeration results from two different people. Approximately every 10th sample was counted by both parties for quality control purposes (see Section 3.4.8 for more details). During the challenge tests, the raw water pH, dissolved oxygen, temperature, turbidity and slow sand filter effluent turbidity were monitored. These results are shown in Appendix E: Table E.1 to E.3.

Subsequent replicate challenge tests on each pilot system have been conducted to obtain results from a more mature system under a range of seasonal conditions and hydraulic loadings, however these results are not discussed here. These tests, which generally showed higher removals than those herein, were conducted by others who continued to operate the pilot plant after data collection for this thesis was complete.

Table 6.1: *Cryptosporidium* Challenge Test Results for Test 5a

Test Info	Sample	Date of processing (2004)	Volume of sample (mL)	Count 1	Count 2	Average count	Conc. (oocyst/L)	% removal†	Log removal†
Test 5a	2* - 0	9-Jun	180		2	2	11		
Pilot 1-T1	2* - 1	9-Jun	1		68	68	68,000		
0.4 m/h$^{\Phi}$	2* - 3	2-Jun	2		706	706	353,000		
31-May	2* - 4	9-Jun	1		140	140	140,000		
	2* - 5	2-Jun	2		349	349	174,500		
	2* - 6	9-Jun	1		83	83	83,000		
	2* - 8	15-Jun	100		1376	1376	13,760		
	3 - 0	9-Jun	250	52		52	208		
	3 - 2	9-Jun	250	22		22	88	99.87	2.89
	3 - 4	2-Jun	250	1		1	4	100.00	4.95
	3 - 5	9-Jun	250	192		192	768	99.45	2.26
	3 - 6	21-Jun	200	96		96	480	99.72	2.56
	3 - 7	9-Jun	250	78		78	312	99.62	2.42
	3 - 9	2-Jun	250	36		36	144	98.95	1.98
							average =	99.60	2.84
							standard deviation =	0.37	1.07

† based on 1 hour detention time in SSF

Φ hydraulic loading rate through slow sand filter

Note: only one count done on each sample in Test 5a

Table 6.2: *Cryptosporidium* Challenge Test Results for Test 5b

Test Info	Sample	Date of processing (2004)	Volume of sample (mL)	Count 1	Count 2	Average count	Conc. (oocyst/L)	% removal†	Log removal†
Test 5b	2* - 0	16-Jun	100		1249	1249	12,490		
Pilot 1-T1	2* - 1	21-Jun	1		37	37	37,000		
0.8 m/h$^{\Phi}$	2* - 3	15-Jun	1	47	67	57	57,000		
7-Jun	2* - 4	16-Jun	1		103	103	103,000		
	2* - 5	22-Jun	1		106	106	106,000		
	2* - 6	16-Jun	1		74	74	74,000		
	2* - 8	16-Jun	100		244	244	2,440		
	3 - 0	21-Jun	200	6		6	30		
	3 - 1.5	21-Jun	200	16		16	80	99.78	2.67
	3 - 3.5	15-Jun	250	92		92	368	99.35	2.19
	3 - 4.5	21-Jun	200	30		30	150	99.85	2.84
	3 - 5.5	22-Jun	200	73		73	365	99.66	2.46
	3 - 6.5	21-Jun	200	9		9	45	99.94	3.22
	3 - 8.5	22-Jun	200	14		14	70	97.13	1.54
							average =	99.29	2.49
							standard deviation =	1.08	0.58

† based on 0.5 hour detention time in SSF
Φ hydraulic loading rate through slow sand filter

Table 6.3: *Cryptosporidium* Challenge Test Results for Test 5c

Test Info	Sample	Date of processing (2004)	Volume of sample (mL)	Count 1	Count 2	Average count	Conc. (oocyst/L)	% removal†	Log removal†	Cumulative % removal	Cumulative log removal	Notes
Test 5c	2* - 0	20-May	200		34	34	170					
Pilot 2	2* - 1	20-May	2		231	231	115,500					
0.4 m/h$^{\Phi}$	2* - 2	10-May	1		162	162	162,000					
28-Apr	2* - 3	28-Apr	2		348	348	174,000					
	2* - 4	10-May	1		288	288	288,000					
	2* - 6	28-Apr	1	252	333	292.5	292,500					
	2* - 8	10-May	1		94	94	94,000					
	3 - 0	20-May	400		0	0	0					
	3 - 1	10-May	490		0	0	0					
	3 - 2	10-May	560	0		0	0					
	3 - 3	28-Apr	505	153		153	303	99.74	2.58			
	3 - 4	10-May	505		77	77	152	99.91	3.03			
	3 - 6	10-May	485	73	81	77	159	99.94	3.26			
	3 - 8	28-Apr	500	260	156	208	416	99.86	2.85			
							average =	99.86	2.93			
							standard deviation =	0.09	0.29			
	4 - 0	20-May	500				0					
	4 - 4	20-May	500	5		5	10	96.70	1.48	99.99	4.06	
	4 - 6	20-May	500	12		12	24	84.88	0.82	99.99	3.86	‡
	4 - 7	20-May	500	44		44	88	44.57	0.26	99.97	3.51	
	4 - 8	20-May	500	75		75	150	63.94	0.44	99.95	3.29	‡
							average =	72.52	0.75	99.97	3.68	
							standard deviation =	23.04	0.54	0.02	0.35	

† based on 2 hour detention time in SSF-1, and 1 hour detention time in SSF-2
‡ for calculation purposes, no detention time through SSF-2 was allowed for in calculation of removal
Φ hydraulic loading rate through slow sand filter

Test 5a, the first challenge test on the pilot system 1 (train 1) slow sand filter, was conducted at a slow sand filtration rate of 0.4 m/h, average influent dissolved oxygen level of 5.9 mg/L, average water temperature of 19^{o}C, and average influent turbidity of 3.5 NTU. Despite a

reduced bed depth of 45 cm, removals of *Cryptosporidium parvum* oocysts ranged from 2 to 5 logs. The average removal was 2.8 logs with a standard deviation of 1. All but one measurement exceeded the current Ontario regulatory requirement for 2 log removal of *Cryptosporidium* (MOE, 2003).

Test 5b, the second challenge test on the pilot system 1 (train 1) slow sand filter, was conducted at a hydraulic loading rate of 0.8 m/h, average influent dissolved oxygen level of 6.7 mg/L, average water temperature of 22^{o}C, and average influent turbidity of 2.1 NTU. Despite its reduced bed depth and increased hydraulic loading rate, removals ranged from 1.5 to 3.2 logs. The average removal was 2.5 logs with a standard deviation of 0.6, a little less than its average removal at 0.4 m/h. Again, all but one measurement exceeded the current Ontario regulatory requirement for 2 log removal of *Cryptosporidium.*

Test 5c, the first challenge test on the pilot system 2 slow sand filters (both slow sand filters in series), was conducted at a hydraulic loading rate of 0.4 m/h, average influent dissolved oxygen level of 10.4 mg/L, average water temperature of 11^{o}C, and average influent turbidity of 0.7 NTU. Removals in slow sand filter 1 ranged from 2.6 to 3.3 logs, and the average removal was 2.9 logs with a standard deviation of 0.3. All measurements exceeded the current Ontario regulatory requirement for 2 log removal of *Cryptosporidium*. Although slow sand filter 1 in pilot 2 had a deeper bed depth, its performance was similar to that of pilot 1 at a hydraulic loading rate of 0.4 m/h. However, this test conducted at a much a lower temperature when the filter was less mature. Performance of pilot system 2 would likely be improved in warmer temperatures and with a more mature slow sand filter.

In pilot system 2, slow sand filter 2 contributed an additional removal ranging from 0.3 to 1.5 logs with an average of 0.8 logs and standard deviation of 0.5. This resulted in a total cumulative removal ranging from 3.3 to 4.1 logs, with an average of 3.7 logs and standard deviation of 0.4. Although slow sand filter 2 contributed significant additional removal, it may not be required with a fully mature slow sand filter 1. However, it may provide extra protection against oocyst breakthrough after cleaning of the first slow sand filter, or at the very least, increased robustness during challenging operating conditions.

Overall, the average removal of *Cryptosporidium parvum* oocysts was greater than 2 logs in all three challenge tests. The slow sand filters in both pilot systems achieved similar results despite pilot system 1 having a shallower bed depth (half the bed depth of the slow sand filter in pilot system 2) and an increased hydraulic loading rate of 0.8 m/h. However, improved performance in pilot system 2 would be expected in warmer temperatures, due to its deeper bed depth. In pilot system 2, slow sand filter 2 was able to contribute a substantial additional removal of *Cryptosporidium*, beyond that provided by slow sand filter 1, resulting in a cumulative removal that was approximately 1 log greater than all the other tests. However, its importance after treatment with a fully mature slow sand filter 1 remained to be determined.

It is important to note that improved removals are expected in a fully mature slow sand filter, and subsequent testing did in fact show this. It would be important to determine if such improved performance would be sustained through the colder winter months, with a fully mature filter.

6.4 CONCLUSIONS

Specific conclusions of the turbidity, coliform bacteria, and *Cryptosporidium* challenge test results were summarized at the end of Sections 6.3.1.1, 6.3.1.2, 6.3.2.1, 6.3.2.2, 6.3.2.3, and 6.3.3. A general listing of the overall findings from these results is given below.

6.4.1 Conclusions from Manual Turbidity Results

6.4.1.1 Pilot System 1

1. During the entire study period, with water temperatures ranging from 2.5 to 17.5^{o}C, average influent turbidity of 15 NTU, a number of runoff events as high as approximately 50 NTU, and a slow sand filtration rate ranging from 0.4 m/h (except for a short period of time at 0.2 m/h) to 0.8 m/h (train 2 was operated 0.8 m/h for a period of 8 weeks), train 1 and train 2 produced effluent turbidities below 1 NTU in 93% and 89% of the measurements, respectively. The average slow sand filter effluent turbidity was 0.4 NTU (standard deviation of 0.34) and 0.44 NTU (standard deviation of 0.38) in train 1 and 2,

respectively, which is lower than the current standard for chemically assisted filtration. This performance was remarkable considering that the system was not fully mature prior to the beginning of the study.

2. In a less mature system operating during colder water temperatures (<5^{o}C), 100% and 86% of the effluent measurements from both trains were below 1 and 0.5 NTU, respectively. Train 1 and 2 produced an average effluent of 0.38 and 0.33 NTU, respectively, at a slow sand filtration rate of 0.4 m/h. The results from Chapter 5 were much better during this period, where approximately 99% of effluent measurements were below 0.5 NTU and the average effluent turbidity was approximately 0.27 NTU.

3. In a more mature system operating during warmer temperatures (>10^{o}C), performance was significantly improved where train 1 consistently produced an effluent below 0.3 NTU at a slow sand filtration rate of 0.4 m/h, and train 2 consistently produced a final effluent below 0.5 NTU at 0.8 m/h, which is remarkable for such a high hydraulic loading rate. These results were similar to those found in Chapter 5, however because those results were drawn from a much larger data set, the majority of the data, but not all of the data, were below the regulatory standards stated above.

4. Performance of the roughing filter in train 1 was significantly improved in warmer water temperatures when the filter was more biologically mature.

5. In a less mature system, reducing the hydraulic loading rate to 0.2 m/h during runoff events proved instrumental in maintaining effluent turbidities below 1 NTU. This result is similar to that found in Chapter 5. It remains to be determined if this would be necessary in a fully mature filter.

6. The presence of pre-ozonation did not have a significant effect of effluent turbidity, however this conclusion was drawn from a relatively small data set collected over a short period time. Pre-ozonation may have a measurable effect on effluent turbidity over the long term, especially in warmer water temperatures.

7. The GAC filter was not a consistent contributor to turbidity removal, although it became more important during runoff events and periods of higher slow sand filter effluent turbidities. However, this was based on limited monitoring in relatively cold water before the GAC filter was fully commissioned (i.e. GAC filter occasionally produced turbidity via GAC fines escaping the filter).

6.4.1.2 Pilot System 2

1. During the entire study period, with water temperatures ranging from 2.5 to 17.5^{o}C, average influent turbidity of 15 NTU, a number of runoff events as high as approximately 50 NTU, and a slow sand filtration rate ranging from 0.4 (except for a short period of time at 0.2 m/h) to 0.6 m/h (for three weeks), pilot system 2 produced effluent turbidities below 1 NTU in 96.5% of the measurements. The average effluent turbidity in slow sand filter 1 and 2 was 0.47 NTU (standard deviation of 0.36) and 0.31 NTU (standard deviation of 0.17), respectively. Effluent turbidity performance, in terms of effluent sensitivity to fluctuations in the influent turbidity and operational disturbances, was better than in pilot 1, likely due to the deeper bed depths in the roughing filter and slow sand filter.

2. In a less mature system operating during colder water temperatures (<5^{o}C), slow sand filter 1 consistently produced an effluent below 1 NTU and 93% of the measurements were below 0.5 NTU, with a slow sand filtration rate of 0.4 m/h. Even better, slow sand filter 2 consistently produced an effluent below 0.5 NTU and 87% of the measurements were below 0.3 NTU. The average effluent turbidity of slow sand filter 1 and 2 was 0.34 and 0.27 NTU, respectively. These results are somewhat better than those found in Chapter 5, where 94% and 82% of effluent measurements were less than 1 NTU and 0.5 NTU, respectively. Nevertheless, this performance is remarkable considering that the system was not fully mature.

3. In a more mature system operating during warmer temperatures (>10^{o}C), performance was significantly improved where pilot 2 (effluent of slow sand filter 1) consistently produced an effluent below 0.3 NTU at a slow sand filtration rate of 0.4 m/h. These

results were similar to that found in Chapter 5, however because those results were drawn from a much larger data set, the majority of the data, but not all of the data, were below the regulatory standard stated above.

4. Performance of roughing filter B in pilot system 2 was significantly better than roughing filter A in pilot system 2 and the roughing filters in pilot system 1. In addition, the performance of roughing filter B was improved in warmer water temperatures when the filter was more biologically mature. However, due to its larger headloss and potential filter cleaning problems, its operational integrity remains in question, and should be monitored over a long-term period of warmer water temperatures.

5. In a less mature system, reducing the hydraulic loading rate to 0.2 m/h during runoff events proved instrumental in maintaining effluent turbidities below 1 NTU. This result is similar to that found in Chapter 5. It remains to be determined if this would be necessary in a fully mature filter.

6. In a less mature system, the second slow sand filter in series was important for meeting the standard of 0.5 NTU during cold water conditions. Its importance in a fully system remains to be determined.

7. The second slow sand filter in series was important for reducing effluent turbidity to below 1 NTU when operating the system at an increased hydraulic loading during runoff events of high turbidity. In fact, at a hydraulic loading of 0.6 m/h, slow sand filter 2 reduced the average slow sand filter 1 effluent from 0.95 NTU to an average of 0.52 NTU. Its importance in a fully mature system remains to be determined.

6.4.2 Conclusions from Total and Fecal Coliform Results

6.4.2.1 Pilot System 1

1. Although the main purpose of pre-ozonation in pilot system 1 was for removal of dissolved organic carbon and colour, it achieved excellent removal of total coliforms to

below detection limits and complete removal of fecal coliforms, including during cold water conditions.

2. Without pre-ozonation and during cold water temperatures (<5^{o}C), train 2 did not achieve complete removals of coliforms, likely due to inadequate biological maturity of the filter and reduced biological treatment, as well as reduced bed depth (~0.5 m) in the slow sand filter. Improved removals are expected in a fully mature filter, however this remains to be determined.

3. Without pre-ozonation and during cold water temperatures, the roughing filter was a significant contributor to coliform removal.

6.4.2.2 Pilot System 2

1. In the entire study, pilot system 2 (operating without pre-ozonation) reduced total and fecal coliforms to ≤2 MPN/100 mL and 0 CFU/100 mL in 80% of the measurements, with influent levels as high as 2400 MPN/100 mL and 1377 CFU/100 mL, periods of water temperature less than 5^{o}C, and a hydraulic loading rate mostly at 0.4 m/h (except for a three period of hydraulic loading rate at 0.6 m/h and a short period of time at 0.2 m/h). Generally, performance was better than in pilot 1 (without pre-ozonation), which may mostly be due to deeper bed depths in the roughing and slow sand filter.

2. During cold water temperatures (<5^{o}C), removal of total coliforms was complete in 3 out of 4 measurements, and during warm water conditions (>9^{o}C) removal was complete in 100% of measurements.

3. Roughing filter B consistently outperformed roughing filter A, in terms of total and fecal coliform removals, likely due to its increased filtration efficiency from deeper bed depths of finer gravel media.

4. On numerous occasions, the second slow sand filter was important for providing additional removals of coliforms during periods of higher influent levels, incomplete

removals in the previous filtration stages, increased hydraulic loadings, and colder water temperatures. Its importance in a more biologically mature system remains to be determined.

6.4.3 Conclusions from *Cryptosporidium* Challenge Tests

1. The average removal of *Cryptosporidium parvum* oocysts was greater than 2 logs, the current Ontario regulatory requirement, in all three challenge tests.

2. Pilot system 1 (train 1), operating at 0.4 m/h, achieved an average removal of 2.8 logs, and only a marginally lower average removal of 2.5 logs when operated at 0.8 m/h, which is remarkable for such a high hydraulic loading rate.

3. Pilot system 2 performed the best with an average removal of 2.9 logs, likely due to its deeper bed depth. For this reason, the slow sand filter in pilot 2 was expected to perform much better than in pilot 1. However, its performance was only marginally better likely because it was tested at a much lower water temperature when it was less biologically mature (pilot 2 was tested at 11^{o}C compared to 19-22^{o}C in pilot 1).

4. In pilot system 2, the second slow sand filter contributed a substantial additional average removal of 0.8 logs, beyond that provided by slow sand filter 1. This resulted in a cumulative average removal of 3.7 logs after two slow sand filters in series, approximately 1 log greater than all the other tests.

5. Improved performance is expected in a fully mature filter, and whether this improved performance could be sustained through the colder winter months remains to be determined.

Chapter 7

7 Conclusions and Recommendations

7.1 CONCLUSIONS

7.1.1 Major Conclusions

In this section, the major conclusions from the results of this study are presented, which are each supported by a listing of relevant minor conclusions that validate each major finding.

1. The performance of both pilot multistage filtration systems was highly dependent on the biological maturity of the system, such that:
    a. with a less mature system operating in cold water conditions (<5^{o}C), effluent turbidity was consistently below 1 NTU and mostly below 0.5 NTU during normal periods of operation (no major runoff events and a hydraulic loading of 0.4 m/h). However, performance of both pilots was particularly good (<0.3 NTU) only during periods of stable influent quality (influent turbidity below 10 NTU) and stable operational conditions (no filter cleanings).
    b. with a less mature system operating in cold water conditions (<5^{o}C), removal of coliform bacteria was incomplete in the absence of pre-ozonation, whereas, with a more mature system operating in warm water conditions (>9^{o}C), removal was complete in all measurements.
    c. with a less mature system operating in cold water conditions (<5^{o}C), a higher sensitivity of effluent turbidity to increased influent turbidity, increased hydraulic loadings (0.6 m/h), and operational disturbances such as filter cleaning, resulted in effluent turbidity occasionally above 1 NTU. However, with a more mature system operating in warm water conditions (7.5 to 17.5^{o}C), effluent turbidity was below 0.3 NTU, even during runoff events of high turbidity (>25 NTU), and sometimes below 0.2 NTU.
    d. with a more mature system operating in warm water conditions (19-22^{o}C), the slow sand filter in pilot 1 (bed depth of 0.5 m) achieved average *Cryptosporidium*

removals of 2.8 and 2.5 logs, when operated at 0.4 and 0.8 m/h, respectively, whereas the filter in pilot 2 (much deeper bed depth of 1 m) achieved only a marginally better removal of 2.9 logs, likely because it was tested when the filter was less mature (temperature was only 11^{o}C at the time of testing). Subsequent testing by others after longer term maturation resulted in average removals of 3 to 4 logs through the slow sand filter (these results were not documented or discussed in this thesis).

2. With a less biologically mature system, the performance of both pilot multistage filtration systems was highly dependent upon the hydraulic loading rate of the system, such that:
    a. in a less mature system operating in cold water conditions (<10^{o}C), reducing the hydraulic loading rate to 0.2 m/h from 0.4 m/h was important for achieving effluent turbidity below 1 NTU during runoff events of high turbidity.
    b. in a less mature system operating in cold water conditions (<5^{o}C), an increased hydraulic loading of 0.6 m/h resulted in effluent turbidity above 1 NTU during runoff events of high turbidity.
    c. in contrast, in a more mature system operating in warm water conditions (7.5 to 17.5^{o}C), increased hydraulic loadings up to 0.8 m/h resulted in good effluent performance (<0.5 NTU), even during runoff events of high turbidity.

3. The second slow sand filter in series provided additional robustness to the process by:
    a. reducing effluent turbidity to below 1 NTU during cold water runoff events of high turbidity and increased hydraulic loadings (0.6 m/h). It should be noted, however, that the importance of a second slow sand filter after treatment in a fully mature system, remains to be determined.
    b. achieving effluent below 0.3 NTU during normal periods of operation,
    c. providing additional removals of coliforms under challenging operating conditions, and
    d. contributing an additional average removal of *Cryptosporidium* of 0.8 logs, which resulted in cumulative removal of 3.7 logs, approximately 1 log greater than all the other challenge tests.

4. The roughing filter was not only important for protecting the slow sand filter from solids loading, but also an important treatment barrier in the overall process of pathogen removal, such that:
    a. during cold water temperatures (<5^{o}C) when the system was less mature, the roughing filter was a significant contributor to coliform removal.
    b. removal of turbidity and coliform bacteria was significantly better in roughing filter B (pilot 2), which had deeper bed depths of finer media, than in roughing filter A (pilot 2). It should be noted, however, that due to headloss buildup and potential filter cleaning problems, the operational integrity of roughing filter B remains in question, and should be monitored over a long-term period of warmer water temperatures.
    c. removal of turbidity in the roughing filter was significantly improved during warmer water temperatures when the filter was more biologically mature. This suggests that biological treatment is an important treatment mechanism in roughing filters.
    d. the roughing filter preserved the run length of the slow sand filter by capturing a large amount of solids before entering the slow sand filter.

5. Pre-ozonation was suspected to be important for enhanced removal of turbidity and was a significant contributor to the removal of coliform bacteria, such that:
    a. enhanced biological growth resulting from pre-ozonation in pilot 1 may have compensated for the potential of reduced treatment capacity resulting from its shallower bed depths. This would explain the fact that effluent turbidity performance was similar between pilot system 1 and 2 (despite pilot 2 having twice the bed depth of pilot 1), except during warmer water conditions, where effluent turbidity from pilot 1 (which had pre-ozonation), was more frequently below 0.3 NTU than in pilot 2 (which did not have pre-ozonation).
    b. pre-ozonation achieved complete removal of coliform bacteria (as would be expected at the applied dose), including during cold water conditions.

7.1.2 Additional Conclusions

1. The presence of pre-ozonation had no statistically significant effect of effluent turbidity, however this testing was conducted over a short period of time in cold water temperatures. Pre-ozonation may have an impact on effluent turbidity during periods of warmer water temperatures, and may have compensated for the potential of reduced treatment capacity resulting from shallow slow sand filter bed depths, however this remains to be determined.

2. In the absence of pre-ozonation, removal of coliform bacteria was generally better in pilot system 2 than in pilot system 1, likely due to its deeper bed depths in the roughing filter and slow sand filter.

3. The design of the roughing filter (in terms of the media size and bed depth of the gravel layers) in pilot 1 and its relatively high hydraulic loadings were suspected as the cause of poor run lengths in the roughing filter and slow sand filter. Other potential factors suspected in causing poor slow sand filter run lengths were increased headloss due to pre-ozonation, inadequate methods of cleaning and removal of headloss causing material from the filter, increased hydraulic loadings through the slow sand filter (0.8 m/h), as well as a number of runoff events of high turbidity.

4. Pilot 2, which had no pre-ozonation, deeper roughing filters, and was not operated at 0.8 m/h, generally had much longer slow sand filter runs. It is recommended that the roughing filter in pilot 1 be modified to a deeper bed depth for increased solids storage capacity and the filter area increased to reduce its range of hydraulic loadings. It is also recommended that the slow sand filter in pilot 1 not be operated at 0.8 m/h for more than one day, which is greater than the time to clean a parallel filter that has been taken offline.

5. There was insufficient evidence to determine whether the granular activated carbon (GAC) filter provided additional removal of turbidity beyond that achieved by the slow sand filter. However, this was based on limited monitoring and the filter was not fully commissioned at the time of testing, thus was occasionally producing turbidity via the escape of GAC fines.

7.2 IMPLICATIONS

The results of this study suggest that multistage filtration can perform well in northern climates with water temperatures ranging down to 2^{o}C, limited filter maturity, elevated raw water turbidity, and increased hydraulic loading rates, particularly with respect to removal of turbidity, coliform bacteria, and *Cryptosporidium*. Furthermore, its ability to meet the current Ontario turbidity regulations and greater than 2 log removal of *Cryptosporidium* over a range of operating conditions, with little or no process adjustment, is a testament to the robustness and minimal maintenance requirements of the process, which are desirable attributes for small water systems that are often located in rural areas.

From the results, it is obvious that the slow sand filter is the most important stage in the multistage filtration process for improving the quality of the effluent. In fact, Wegelin (1988) states, "no other single water treatment process can improve the physical, chemical, and bacteriological water quality of surface water better than slow sand filtration". However, this research also demonstrated the importance of roughing filtration, and how it was integral to the overall process from both an operative and treatment standpoint. In this study, the roughing filter fulfilled its traditional role of protecting the slow sand filter from solids overloading and preserving its filter run length, especially during numerous runoff events of influent turbidity as high as 100 NTU and increased hydraulic loadings. However, it was also an important treatment barrier in the overall multistage filtration process, in terms of the removal of turbidity and coliform bacteria, by providing increased surface area for sedimentation and biological activity. It was especially important during colder water temperatures, when the system was less biologically mature, and contributed to the removal of coliform bacteria in the absence of pre-ozonation.

The design of the roughing filter had a substantial impact on the removal of turbidity and total coliforms, where roughing filter B in pilot system 2 outperformed all other roughing filters, due to its increased filtration efficiency increased bed depths of finer gravel media. However, it is important to ensure that increasing the filtration efficiency of a roughing filter will not affect its operational integrity over long term operation. One of the most important

advantages of the upflow roughing filter is that it can be cleaned by gravity flushing, where high drainage velocities dislodge and flush solids from the filter, without the use of energy-intensive backwashing. However, a roughing filter with a deeper bed depth of finer media may eventually become clogged, especially after long term biological maturity of the filter, resulting in a sufficiently high headloss to prevent the high drainage velocities that are required to efficiently flush the filter clean. Thus, it remains to be seen whether, with long term maturation, it will continue to operate within a reasonable range of headloss, and continue to attain sufficient drainage velocities during cleaning practices.

On numerous occasions throughout this study, the biological maturity of the system has arisen as paramount to achieving a stable effluent quality, despite large fluctuations in influent water quality and operational conditions. However, in a less biologically mature system, the bed depth of the roughing and slow sand filters and their respective hydraulic loading rates were key factors in attaining good effluent quality. In addition, the second slow sand filter in series was important, especially in a less mature system, for providing increased robustness to the overall process, and protecting against turbidity and coliform breakthrough.

Overall, each individual stage of the multistage system was an important treatment barrier in the process of turbidity and pathogen removal. Collectively, the entire system provides a robust, multi-barrier method of drinking water treatment that is both reliable and cost-sustainable for small communities. Further work to define the operational envelope in which multistage filtration can perform satisfactorily for small systems in North America is warranted, in particular, its performance in colder temperatures with a fully mature filter. In addition, further work will be important to determine the optimal hydraulic loading rate for a given range of raw water quality conditions and temperatures.

While this research demonstrated the performance of multistage filtration using pilot scale testing, it is important to note that full-scale plants tend to produce significantly better results than pilot facilities. This is likely because pilot facilities are usually not in operation long enough to experience improved performance from long term biological maturation of the filter bed, similar to that experienced by full-scale plants. For example, the North Haven,

Maine pilot, constructed by MS Filter Inc., only achieved effluent turbidities of 0.4 NTU in a 9-month test with raw water turbidities ranging from 1 to 5 NTU. However, the full-scale plant in the first year produced turbidities less than 0.2 NTU, and less than 0.1 NTU in the second year (LeCraw, 2004). Thus, small pilot facilities may not exactly simulate larger filter performance, and progressive improvement in performance can typically be expected from full-scale multistage filtration systems until they are fully mature.

Overall, multistage filtration is a sustainable and cost-effective technology that, through this research, appears to be a safe, reliable, and robust treatment alternative for small and non-municipal water systems in North America and the developing world. Further, based on its performance with challenging influent water quality and cold water conditions, multistage filtration holds particular promise for small communities in northern climates that are required to meet safe drinking water regulations, but are dependent on surface water sources of variable water quality and temperatures.

7.3 RECOMMENDATIONS

7.3.1 Further Research with Multistage Filtration on the Grand River

This research demonstrated that multistage filter performance improves with time as the biological maturity of the system increases. Thus, there are a number of recommendations for future research which involve testing multistage filter performance with a fully mature filter, which is typical of a full-scale system, especially during increased influent turbidity, high hydraulic loadings, and cold water temperatures. These, along with other questions that have arisen from this research, are presented in the following list.

1. Determine whether, during cold water temperatures, there is relatively less sensitivity of effluent turbidity to increased influent turbidity and operational disturbances (such as cleaning and temporary surges in hydraulic loading rate) with a fully mature filter, compared to a less mature filter.

2. Determine the performance of a fully mature system with increased influent turbidity and cold water temperatures at increased hydraulic loadings (up to 0.8 m/h). In particular, determine whether an effluent turbidity consistently less than 1 NTU can be achieved with long term operation at a higher hydraulic loading rate of 0.6 m/h.

3. Determine whether, in a fully mature system, reducing the hydraulic loading rate to 0.2 m/h during cold water events of high turbidity is required for meeting the current Ontario standard of 1 NTU for slow sand filter effluent.

4. Determine whether a fully commissioned GAC filter (ie. no escape of GAC fines) is a significant contributor to turbidity and coliform removal.

5. Determine whether, in a fully mature system, a second slow sand filter in series is required for meeting the standard of 0.5 NTU in cold water conditions, and for achieving effluent turbidity below 1 NTU during cold water events of high influent turbidity combined with a higher hydraulic loading of 0.6 m/h.

6. Further investigate whether the design of the roughing filter (ie. reduced bed depth and media size), higher hydraulic loadings, and pre-ozonation have a negative impact on the filter run length of roughing filters and slow sand filters.

7. Investigate whether the roughing filter in pilot 1 can be modified to a deeper bed depth for increased solids storage capacity and the filter area be increased to reduce its range of hydraulic loadings to improve particle sedimentation and capture in the media.

8. Characterize the settling velocities of raw water particulate matter in the Grand River and estimate optimal roughing filter design using models developed by Collins et al. (1994b).

9. Examine the impact of media size and hydraulic loading rate on the removal of colloidal matter in roughing filters.

10. Determine whether roughing filter B, which has deeper bed depths of finer gravel media, will continue to operate within a reasonable range of headloss, and continue to attain sufficient drainage velocities during cleaning.

11. Determine whether, in a fully mature system, the second slow sand filter is required for providing additional removals of coliforms during periods of high influent levels, increased hydraulic loadings, and colder water temperatures.

12. Determine whether improved removals of *Cryptosporidium* can be achieved in a fully mature filter, including during colder water conditions.

13. Determine the removal of *Cryptosporidium* in roughing filters to define the log removal credits that can be attributed to roughing filtration.

14. Systematically investigate the impact of slow sand filter cleaning practices on effluent turbidity, the recovery of filtration efficiency after cleaning, and the importance of post-treatment after cleaning.

15. With the goal of reducing the filter area (footprint) of the second slow sand filter in series to a practical size for full-scale installations, determine its performance under long term operation at high hydraulic loading rates (0.6 to 0.8 m/h).

16. Investigate the impact of post-treatment with a GAC sandwich filter (sand-GAC-sand), which is essentially a combination of a slow sand filter and GAC filter.

17. Investigate the impact of a 'dynamic' roughing filter (Wegelin, 1996) prior to the multistage filter to protect the system from solids overloading during extreme events of high turbidity.

18. Investigate the removal of organic matter, colour, and disinfection by-product formation potential in both pilot multistage systems.

19. Due to the variable operating conditions (hydraulic loadings and influent turbidity), it may be useful to normalize the filter run length of the roughing filter and slow sand filter by the total volume of water treated or total mass of turbidity-related particulate matter removed.

7.3.2 Operation of Multistage Filters

Based on the conclusions of this research, a number of operational recommendations for optimizing performance of multistage filters are given in the following list.

1. It is recommended that start-up of a multistage filtration system be done in warmer water temperatures, ideally at the beginning of the warm season, to achieve full biological maturation of the filter bed prior to the beginning of the cold season.

2. In a less mature filter, with cold water temperatures (<5^{o}C) and a hydraulic loading rate of 0.4 m/h, assurance of effluent turbidity below 1 NTU can only be expected if influent turbidity is less than approximately 10 NTU (for a shallow slow sand bed depth ~0.5m) or 20 NTU (for a deep slow sand bed depth ~1m). If influent turbidity is consistently higher than these levels, reducing the hydraulic loading rate to 0.2 m/h is recommended until the filter is fully mature.

3. Reducing the hydraulic loading rate to 0.2 m/h is recommended for achieving effluent turbidity below 1 NTU during cold water runoff events of high turbidity (over 50 NTU). Otherwise, post treatment with a second slow sand filter in series is recommended.

4. In a less mature filter, when operating a multistage filter at a higher hydraulic loading rate of 0.6 m/h, an effluent turbidity consistently below 1 NTU can only be expected with influent turbidity below approximately 15 NTU. After the system becomes more acclimatized to the increased loading conditions, an effluent below 1 NTU may be achieved with somewhat higher influent turbidity (perhaps up to 25 NTU).

5. It is not recommended to operate a multistage filter at a hydraulic loading rate of 0.6 m/h or higher during runoff events of high turbidity and cleaning events. Otherwise, post treatment with a second slow sand filter in series is recommended to achieve effluent below 1 NTU.

6. Due to dramatically reduced filter run times, consistently operating the slow sand filter at hydraulic loading rate of 0.6 m/h is not recommended unless effluent turbidities are below 10 NTU. However, further testing is required to confirm the validity of this recommendation.

7. Due to dramatically reduced filter run times, it is recommended that the slow sand filter not be operated at 0.8 m/h for more than one day, which is typically the time it takes to clean a parallel filter that has been taken offline for cleaning.

8. A fully biologically mature multistage filter is recommended for achieving good removals of coliform bacteria during colder water temperatures, without the use of pre-ozonation.

9. Deeper bed depths (~1 m) in the roughing filter and slow sand filter are recommended for achieving good removals of coliform bacteria during colder water temperatures, without the use of pre-ozonation.

10. A second slow sand filter in series is recommended for providing additional removals of coliforms in systems that are not fully mature, especially during periods of higher influent levels, increased hydraulic loadings, and colder water temperatures.

11. A deeper bed depth in the slow sand filter (~1 m) is recommended for enhanced removal of *Cryptosporidium* in a less mature system that is operating in cold water temperatures. However, based on the results of this study, at least 2 logs removal can still be expected in a less mature filter with a bed depth of 0.5 m.

12. A second slow sand filter in series is recommended for achieving over 3 logs removal of *Cryptosporidium* in a less mature system that is operating in cold water temperatures. However, it is expected that a second slow sand filter is not required to achieve this in a fully mature system, and further testing is required to confirm this.

Appendix A: The Impact of L/d Ratio on the Design of Roughing Filters

The following formula, from Iwasaki (1937), describes the removal of particles in a filter media due to filtration mechanisms:

$$\frac{N}{N_o} = e^{-\eta\frac{L}{d}} \qquad \text{(Equation A.1)}$$

where, N = number of particles at depth L

N_o = number of particles in influent

η = collection efficiency

L = depth of bed (m)

d = diameter of media (m)

According to this equation, as the L/d ratio increases, the removal of particles or filtration efficiency increases. This theory has been validated in a study by Wegelin et al. (1986), which shows increasing removal of particles with decreasing gravel size.

Based on this theory, the L/d ratio for a typical roughing filter, roughing filter A in pilot system 2, was calculated. This was done by summing the L/d ratio of each gravel layer in the roughing filter, resulting in a L/d ratio ranging from 90 to 144 (see Table A.1).

Table A.1: Calculation of L/d Ratio for Roughing Filter A

Media	d_{min} (mm)	d_{max} (mm)	L (m)	L/d_{min}	L/d_{max}
Coarse	12.7	19.1	0.5	39.4	26.2
Medium	9.5	12.7	0.4	42.1	31.5
Fine	4.8	9.5	0.3	62.5	31.6
Total			1.2	144.0	89.3

The L/d ratio of a roughing filter is dramatically smaller than that of rapid sand filtration (~1200) and slow sand filtration (~3000). This possibly explains why sedimentation is the dominant mechanism of removal in roughing filters, rather than straining or interception.

In addition, if the L/d ratio of a roughing filter is used to find an equivalent depth of slow sand filter media, the roughing filter has the same filtration capacity of a slow sand filter with a depth of approximately 3 to 5 cm. However, most slow sand filters have a depth of 100 cm, thus the filtration efficiency of a roughing filter does not compare to that of a slow sand filter.

However, a roughing filter with a higher filtration efficiency may be an important treatment barrier in the overall process of pathogen removal in the multistage filter, beyond just its role of protecting the slow sand filter from solids loading. Thus, roughing filter B in pilot system 2 was designed to have a higher filtration efficiency in terms of L/d ratio. This was done by maximizing the bed depth of the smaller media layers in the roughing filter.

The first step in optimizing the L/d ratio of roughing filter B was to omit the coarse media layer (12.7-19.1 mm diameter), which was the largest media size used in roughing filter A. This media had the lowest L/d ratio compared to the other media layers and offered the least removal efficiency in terms of filtration mechanisms. In addition, a layer of extra fine gravel (1.5-3.2 mm) was added to the roughing filter. Subsequently, the next step to maximizing the L/d ratio of roughing filter B was to increase the depth of the smaller media layers. Based on this criterion, the following media configuration was arbitrarily chosen: 0.1 m depth of 9.5-12.7 mm gravel, 0.55m depth of 4.8-9.5 mm gravel, and 0.55m depth of 1.5-3.2 mm gravel (see Figure 3.10). In this configuration, the total L/d ratio for roughing filter B ranged from 238 to 492, as shown in Table A.2, a 166% to 241% increase from the typical media configuration in roughing filter A.

Table A.2: Calculation of L/d Ratio for Roughing Filter B

Media	d_{min} (mm)	d_{max} (mm)	L (m)	L/d_{min}	L/d_{max}
Medium	9.5	12.7	0.1	10.5	7.9
Fine	4.8	9.5	0.55	114.6	57.9
Extra Fine	1.5	3.2	0.55	366.7	171.9
Total			1.2	491.8	237.6

The next step in the design of roughing filter B was to determine whether this media configuration was theoretically practical. One issue to consider is whether it is appropriate to apply the filtration theory to roughing filtration, in addition to the already proven sedimentation theory. The main problem here is that the filtration equation (Equation A.1) generally assumes that the collection efficiency (η) of the media is constant for the entire depth of the filter bed. However, in the case of roughing filtration, growth of biofilm on the media decreases the interstitial pore space in the media, which would increase the collection efficiency of the media. Once a filter bed is fully acclimatized and the biological population has reached a steady state, it is expected that the collection efficiency would be constant only if the influent water quality, specifically the concentration of nutrients and organic matter (ie. food source for bacteria), remained relatively constant throughout the life of the filter. Unfortunately, most surface water sources, such as the Grand River, experience large fluctuations in water quality, which affects the biomass dynamics in the filter media. Thus, the collection efficiency of the filter media cannot necessarily be assumed constant.

Another factor that can affect the collection efficiency is the non-constant concentration of biomass throughout the depth of the filter bed. Since the availability of nutrients is much higher in upstream portions of the filter bed compared to downstream portions, the growth of the biomass is not consistent throughout the entire depth of the filter, thus collection efficiency cannot be assumed constant throughout the filter. Thus, due to the dynamic nature of biomass in the filter, it is important to use caution when applying the filtration theory to roughing filters.

Another issue of practical importance is whether the media configuration in roughing filter B can be successfully backwashed by the rapid drain procedure. This method involves allowing the filters to drain freely by gravity, which induces a very high interstitial velocity in the

media pores that dislodges captured solids and flushes them to waste. Although the recommended drainage velocity is 30 m/h (Wegelin, 1996), some sources suggest velocities as low as 4 to 6 m/h (IRC, 1989). In Table A.3, the average drainage velocity of roughing A and B were 23 and 15 m/h, which was well above the recommendation of 4 to 6 m/h. Thus, the media configuration in roughing filter B, which utilized deeper bed depths of finer media, was sufficient to allow adequate drainage velocities during cleaning of the roughing filter.

Table A.3: Drainage Velocities during Cleaning of Roughing Filter and B

Date	Roughing Filter A				Roughing Filter B			
	Volume (L)	Time (s)	Area (m^2)	Drainage Rate (m/h)	Volume (L)	Time (s)	Area (m^2)	Drainage Rate (m/h)
2-Jul	9	45	0.03195	22.5	7.5	60	0.03195	14.1
12-Jul	9	42	0.03195	24.1	9	60	0.03195	16.9
12-Jul	9	44	0.03195	23.0	9	70	0.03195	14.5
			Average:	23.2			Average:	15.2

The third issue is to determine whether the extra fine media is large enough to allow the roughing filter to perform its main mechanism of sedimentation. If the pore size of the finer media is too small, suspended particles will instead be removed by straining and could result in frequent clogging and headloss development in the filter. Thus, it is important to compare the pore size of the media to the particle size distribution of the raw water.

The minimum pore diameter of the extra fine gravel (1.5-3.2 mm) was 230 μm. Since the majority of suspended particles in the Grand River are less than 30 μm in diameter (see Chapter 4), this media should not clog due to straining of particulates. However, since the media is finer than the minimum roughing filter media diameter of 4 mm recommended by Wegelin (1996), a higher rate of headloss is expected. However, as discussed in Section 5.3.3, the headloss of roughing filter B did not exceed 20 cm throughout the entire study.

The fourth issue to consider is whether sedimentation will in fact occur in the extra fine gravel layer. Sedimentation in filters is similar to conventional settling in a settling tank or clarifier, however, instead of particles settling on the bottom of a tank, the entire upward facing surface area of the filter media is utilized as a settling surface. In roughing filters, Collins et al.

(1994b) defined sedimentation as the main mechanism of removal for particle sizes greater than 1 μm. Thus, it is important that the roughing filter be operated under conditions that allow sedimentation to occur.

According to Wegelin (1996), laminar flow conditions are required to achieve adequate removal of solids by sedimentation. Reynold's number is used to characterize flow conditions, and a Reynold's number less than approximately 10 will ensure laminar flow. In Figure 2.6 in Chapter 2, the removal of turbidity increases as Reynold's number decreases, and greater than 40% removal of turbidity can only be achieved if Reynold's number is less than 10.

Reynold's number is described in Montgomery (1985) by the following relationship:

$$R = \frac{\rho V D}{\mu} \qquad \text{(Equation A.2)}$$

where ρ = density of fluid

μ = viscosity of fluid

D = diameter of particle

V = velocity of fluid

According to Equation A.2, the maximum allowable velocity in the extra fine media (3.2 mm) before laminar flow transitions into turbulent flow is 4 m/h. This assumes a media diameter of 3.2 mm, media porosity of 0.4, water temperature of 25°C, and a viscosity of 0.89×10^{-3} $N \cdot s/m^2$. Since the range of hydraulic loading rates tested in this study was below 0.8 m/h, the extra fine media in roughing filter B did not experience turbulent flow during normal operation, thus sedimentation was expected to occur.

Overall, by applying the filtration theory to roughing filtration, roughing filter B was designed to have a much higher L/d ratio than roughing filter A, thus was expected to perform better than roughing filter A. As discussed in the results of Chapter 6, roughing filter B outperformed roughing filter A in the removal of turbidity and coliform bacteria. In addition,

it operated with minimal headloss and cleaning of the filter by rapid drainage was not a problem. Thus, optimizing the L/d ratio of roughing filters seems to have had a positive effect on roughing filter performance without compromising its operational integrity. The water quality of the influent should be considered during the design of any roughing filter, and pilot testing is recommended to determine the performance of optimized roughing filter designs, before they are put into full-scale practice.

Appendix B: Online Turbidity Data Analysis and Filter Run Length Data

Table B.1: Statistical Analysis of Online Turbidity Data – Pilot 1

Feb 9 to Apr 9	Raw	Train 1	Train 2
average	14.037	0.428	0.450
stdev	18.313	0.406	1.586
n	4016	4417	4347
% frequency (<0.3001)		57.8	59.6
% frequency (<0.5001)		76.3	75.2
% frequency (<1.0001)		92.7	90.1

Feb 9 to Mar 1	Raw	Train 1	Train 2
average	3.158	0.258	0.275
stdev	2.105	0.165	2.237
n	2054	2054	2054
% frequency (<0.3001)		84.4	88.2
% frequency (<0.5001)		99.6	98.7
% frequency (<1.0001)		99.8	99.5

Mar 2 to Mar 6	Raw	Train 1	Train 2
average	32.323	0.632	0.760
stdev	31.447	0.324	0.669
n	399	436	436
% frequency (<1.0001)		97.7	82.6

Mar 2 to Mar 11	Raw	Train 1	Train 2
average	30.468	0.526	0.567
stdev	23.364	0.307	0.553
n	901	651	774
% frequency (<1.0001)		98.5	89.9

Mar 12 to Mar 15	Raw	Train 1	Train 2
average	10.413	0.218	0.223
stdev	1.947	0.012	0.011
n	158	341	341
% frequency (<0.3001)		100.0	100.0

Mar 12 to Mar 21	Raw	Train 1	Train 2
average	9.401	0.336	0.421
stdev	1.944	0.167	0.180
n	296	893	893
% frequency (<0.5001)		78.3	58.7

Mar 15 to Mar 21	Raw	Train 1	Train 2
average	8.242	0.408	0.543
stdev	1.122	0.176	0.115
n	138	552	552
% frequency (<0.5001)		98.7	99.8

Table B.2: Statistical Analysis of Online Data – Pilot 1 (cont'd)

Feb 25 to Mar 22	Raw	Train 1	Train 2	Difference
average	19.428	0.384	0.474	0.093
stdev	20.441	0.234	2.117	2.169
n	1789	2163	2286	2162
			null hypothesis	u1-u2=0
		t calc (=mean/stdev*sqrt(n))		2.000
			t stat (n-1, 0.01)	2.576
			reject null hypothesis?	no

Mar 21 to Apr 4	Raw	Train 1	Train 2
average	24.696	0.727	0.721
stdev	15.523	0.561	0.442
n	1339	1424	1231
% frequency (<1.0001)		77.8	72.2

Apr 9 to Jun 1	Raw	Train 1	Train 2
average	6.674	0.174	0.209
stdev	8.168	0.246	0.097
n	5136	5137	5138
% frequency (<1.0001)		99.2	99.9
% frequency (<.5001)		99.1	98.9
% frequency (<.3001)		97.7	90.6

Apr 9 to Jun 1 (omitted data)	Raw	Train 1	Train 2
average	6.674	0.174	0.228
stdev	8.168	0.246	0.101
n	5136	5137	3983
% frequency (<1.0001)		99.2	99.8
% frequency (<.5001)		99.1	98.5
% frequency (<.3001)		97.7	87.9

Apr 21-29, May 3-25 (T2 @ 0.8 m/h)	Raw	Train 1	Train 2	Difference
average	6.252	0.188	0.277	-0.101
stdev	8.380	0.319	0.115	0.118
n	2908	2907	1871	1870
# frequency (<.5001) - Apr21-29		784.0	577.0	
# frequency (<.5001) - May3-25		2078.0	1236.0	
% frequency (<.5001)		98.5	96.9	
			null hypothesis	u1-u2=0
		t calc (=mean/stdev*sqrt(n))		-37.086
			t stat (n-1, 0.01)	2.576
			reject null hypothesis?	yes

Table B.3: Statistical Analysis of Online Data – Pilot 2

Feb 27 to Mar 15	Raw	Effluent
average	22.228	0.458
stdev	21.915	0.494
n	1398	1615
% frequency (<0.3001)		42.7
% frequency (<0.5001)		82.2
% frequency (<1.0001)		94.3

Mar 19 to Apr 5 (0.6 m/h)	Raw	Effluent
average	23.509	0.776
stdev	15.630	0.273
n	1436	1609
% frequency (<0.3001)		0.0
% frequency (<0.5001)		13.1
% frequency (<1.0001)		82.2

April 13 to June 1	Raw	Effluent
average	6.868	0.249
stdev	8.458	0.045
n	4752	3966
% frequency (<0.3001)		87.9
% frequency (<0.5001)		100.0

Table B.4: Paired T-test: Comparison of Effluent Turbidity Performance of Pilot 1 (Train 1) and Pilot 2 during Periods of Similar Hydraulic Loading Conditions

Overall	Raw	Pilot 1 (Train 1)	Pilot 2	Difference
average	9.558	0.264	0.316	0.051
stdev	15.131	0.323	0.307	0.393
n	3709	4192	4192	4192
			null hypothesis	u1-u2=0
		t calc (=mean/stdev*sqrt(n))		8.452
			t stat (n-1, 0.05)	1.96
		reject null hypothesis?		yes

Colder water conditions	Raw	Pilot 1 (Train 1)	Pilot 2	Difference
average	22.674	0.443	0.502	0.059
stdev	25.016	0.268	0.513	0.527
n	765	1244	1244	1244
			null hypothesis	u1-u2=0
		t calc (=mean/stdev*sqrt(n))		3.918
			t stat (n-1, 0.05)	1.96
		reject null hypothesis?		yes

Warmer water conditions	Raw	Pilot 1 (Train 1)	Pilot 2	Difference
average	6.149	0.189	0.237	0.048
stdev	8.345	0.314	0.045	0.321
n	2944	2948	2948	2948
			null hypothesis	u1-u2=0
		t calc (=mean/stdev*sqrt(n))		8.182
			t stat (n-1, 0.05)	1.96
		reject null hypothesis?		yes

Table B.5: Filter Run Length Data for the Roughing Filters in Pilot System 1 and 2

System	Date	Water Temp. (°C)	Hydraulic Loading Rate* (m/h)	Filter run length since last cleaning (days)
Pilot 1 - Train 1/Train 2	Feb-14	~3.5	1.5	15 days
Pilot 1 - Train 1/Train 2	Feb-16	2.3	1.5	2 days
Pilot 1 - Train 1/Train 2	Mar-06	~4.2	1.5	16 days
Pilot 1 - Train 1/Train 2	Mar-08	4.4	0.75	2 days
Pilot 1 - Train 1/Train 2	Mar-12	~5	0.75	4 days
Pilot 1 - Train 1/Train 2	Mar-22	~2.9	0.75 to 1.5	10 days
Pilot 1 - Train 1/Train 2	Mar-29	8.4	1.5	7 days
Pilot 1 - Train 1/Train 2	Apr-02	~7.5	0.75	4 days
Pilot 1 - Train 1/Train 2	Apr-16	~12	0.75 to 1.5 / 0.75 to 1.5 to 2.4	14 days
Pilot 1 - Train 1/Train 2	Apr-20	~15	1.5 / 2.4	4 days
Pilot 1 - Train 1/Train 2	Apr-21	~13	1.5 / 2.4	1 day
Pilot 1 - Train 1/Train 2	Apr-28	~12	1.5 / 3.0	7 days
Pilot 1 - Train 1/Train 2	May-05	~14	1.5 / 3.0 to 2.4 to 3.0	7 days
Pilot 1 - Train 1/Train 2	May-13	~15	1.5 / 3.0	8 days
Pilot 1 - Train 2	May-16	~17	1.5 / 3.0	3 days
Pilot 1 - Train 1/Train 2	May-21	~17.5	1.5 / 3.0	5 days
Pilot 1 - Train 1/Train 2	May-25	~17	1.5 / 3.0	4 days
Pilot 1 - Train 1/Train 2	May-30	~17.5	1.5	5 days
Pilot 1 - Train 1/Train 2	Jun-03	~17.5	1.5	4 days
Pilot 2	Mar-08	5	0.9	37 days
Pilot 2	Mar-12	~5	0.9 to 0.5	4 days
Pilot 2	Mar-22	4.8	0.5 to 0.9 to 1.35	10 days
Pilot 2	Mar-29	9.4	1.35	7 days
Pilot 2	Apr-02	~8	1.35	4 days
Pilot 2	Apr-16	~10	1.35 to 0.9	14 days
Pilot 2	Apr-26	11.2	0.9	10 days
Pilot 2	May-13	~15	0.9	17 days
Pilot 2	May-17	17.4	0.9	4 days
Pilot 2	May-25	~17.5	0.9	8 days
Pilot 2	Jun-03	~17.5	0.9	9 days
			average (pilot 1)	6.42
			stdev (pilot 1)	4.43
			average (@0.75 to 1.5 m/h)**	6.90
			stdev (@0.75 to 1.5 m/h)**	5.11
			average (@2.4 to 3.0 m/h)**	4.88
			stdev (@2.4 to 3.0 m/h)**	2.36
			average (pilot 2)	11.27
			stdev (pilot 2)	9.48
			average (@0.5 to 0.9 m/h)**	13.17
			stdev (@0.5 to 0.9 m/h)**	12.61
			average (@1.35 m/h)**	5.50
			stdev (@1.35 m/h)**	2.12

Note: In pilot system 1, when one train required cleaning, the other train was usually also cleaned to maintain consistency between both trains, even if it was not required.

* includes all hydraulic loading conditions since the previous cleaning

** filter runs that included a transition from one hydraulic loading to another, were not included in the calculations for 'average' or 'stdev'.

Table B.6: Filter Run Length Data for Slow Sand Filters in Pilot System 1 and 2

System	Date	Water Temp. (°C)	Hydraulic Loading Rate* (m/h)	Method of Cleaning	Filter run length since last cleaning (days)
Pilot 1 - Train 1/Train 2	Feb-14	~3.5	0.4	unknown	15 days
Pilot 1 - Train1/Train 2	Feb-16	2.3	0.4	wet harrow	2 days
Pilot 1 - Train 1/Train 2	Feb-20	4	0.4	scraping	4 days
Pilot 1 - Train 1/Train 2	Feb-21	~4	0.4	upflow	1 day
Pilot 1 - Train 1	Mar-15	5.3	0.2 to 0.4	unknown	22 days
Pilot 1 - Train 1/Train 2	Mar-24	~5	0.4	scraping/upflow	9 days / 31 days
Pilot 1 - Train 1/Train 2	Apr-16	~12	0.2 to 0.4 / 0.2 to 0.4 to 0.6	unknown	23 days
Pilot 1 - Train 1/Train 2	Apr-21	~13	0.4 / 0.6	upflow	5 days
Pilot 1 - Train 1/Train 2	Apr-28	~12	0.4 / 0.8	unknown	7 days
Pilot 1 - Train 1/Train 2	May-05	~14	0.4 / 0.6 to 0.8	scraping	7 days
Pilot 1 - Train 2	May-07	14.7	0.8	upflow	2 days
Pilot 1 - Train 1/Train 2	May-13	~15	0.4 / 0.8	upflow	6 days
Pilot 1 - Train 2	May-16	~17	0.8	upflow	3 days
Pilot 1 - Train 1/Train 2	May-21	~17.5	0.4 / 0.8	upflow	5 days
Pilot 2	Mar-12	~5	0.4 to 0.2	scraping	83 days
Pilot 2	Apr-02	~8	0.2 to 0.4 to 0.6	scraping	21 days
Pilot 2	Apr-16	~10	0.6 to 0.4	scraping	14 days
Pilot 2	May-17	17.4	0.4	scraping	31 days
				average (pilot 1)	7.93
				stdev (pilot 1)	7.11
				average (@0.2 to 0.4 m/h)	8.83
				stdev (@0.2 to 0.4 m/h)	8.28
				average (@0.6 to 0.8 m/h)	5.00
				stdev (@0.6 to 0.8 m/h)	1.91
				average (pilot 2)	37.25
				stdev (pilot 2)	31.29
				average (@0.2 to 0.4 m/h)	57.00
				stdev (@0.2 to 0.4 m/h)	36.77

Note 1: any effluent turbidity spike above 1 NTU immediately returned to less than 1 NTU within 2 to 3 hours

Note 2: In pilot system 1, when one train required cleaning, the other train was usually also cleaned to maintain consistency between both trains.

*includes all hydraulic loading conditions since the previous cleaning

** filter runs that included a transition from one hydraulic loading to another, were not included in the calculations for 'average' or 'stdev'.

Appendix C: Manual Handheld Turbidity Data and Analysis

Table C.1: Paired T-test: Determination of Difference in Influent Handeld Turbidity Measurements Between both Pilot Systems

	Influent Turbidity (NTU)		
Date	**Pilot 1**	**Pilot 2**	**Difference**
2-Feb	2.4	1.57	0.83
9-Feb	1.7	1.55	0.15
11-Feb	1.91	2.82	-0.91
13-Feb	2.07	2.1	-0.03
16-Feb	1.78	2.01	-0.23
18-Feb	2.27	2.01	0.26
20-Feb	2.78	2.27	0.51
23-Feb	3.08	2.74	0.34
25-Feb	2.66	2.36	0.3
1-Mar	4.58	4.14	0.44
3-Mar	27.4	22	5.4
5-Mar	33.8	43.1	-9.3
8-Mar	48.3	44	4.3
12-Mar	22	16.1	5.9
15-Mar	15.8	9.43	6.37
17-Mar	7.26	6.25	1.01
22-Mar	20.2	17.3	2.9
26-Mar	44.4	56.3	-11.9
29-Mar	41.6	29.3	12.3
31-Mar	47.5	23.1	24.4
5-Apr	12.6	11.1	1.5
7-Apr	5.8	11	-5.2
12-Apr	5.72	4.17	1.55
14-Apr	3.88	2.63	1.25
19-Apr	29.9	22.8	7.1
26-Apr	3.34	2.95	0.39
10-May	17.8	11.6	6.2
		average	2.068
		stdev	6.553
		n	27
		null hypothesis	u_1-u_2=0
		t calc (=mean/stdev*sqrt(n))	1.640
		t stat (n-1, 0.025)	2.056
		reject null hypothesis?	no

Table C.2: Comparison between Online and Manual Handheld Turbidimeters

Pilot System 1: Train 1

		Turbidity (NTU)		
Date	Time	Online	Manual	Difference
Jul-16	12:00	0.181	0.2	0.019
Jul-20	10:40	0.149	0.16	0.011
Jul-23	4:15	0.145	0.17	0.025
Jul-27	10:20	0.082	0.11	0.028
Jul-29	10:20	0.151	0.12	-0.031
	Average	0.142	0.152	0.010
	Stdev	0.036	0.037	0.024

Pilot System 1: Train 2

		Turbidity (NTU)		
Date	Time	Online	Manual	Difference
Jul-16	12:00	0.199	0.21	0.011
Jul-20	10:40	0.184	0.21	0.026
Jul-23	4:15	0.174	0.2	0.026
Jul-27	10:20	0.137	0.17	0.033
	Average	0.174	0.198	0.024
	Stdev	0.026	0.019	0.009

Pilot System 2

		Turbidity (NTU)		
Date	Time	Online	Manual	Difference
Mar-01		0.195	0.22	0.025
Jul-16	1:05	0.181	0.21	0.029
Jul-27	11:15	0.108	0.13	0.022
	Average	0.161	0.187	0.025
	Stdev	0.047	0.049	0.004

Table C.3: Manual Handheld Turbidity Data – Pilot 1 (Train 1)

			Train 1																		
Date	Time	Temp (°C)	Flow (Lpm)	Turb. (B)	Turb. (0)	Turb. (1)	Turb. (2)	Log rem. (2)	Turb. (3)			ave.	stdev	Log rem. (3)	cum. Log Rem. (3)	Turb. (4)			ave.	log rem. (4)	cum. Log rem. (4)	comments
2-Feb	18:00		0.3	0.05	2.4	3.86	0.86	0.446	0.22	0.22	0.31	0.25	0.05	0.54	0.98						0.98	
9-Feb	14:00	7.3	0.4	0.08	1.7	1.95	0.89	0.281	0.2	0.21	0.27	0.23	0.04	0.59	0.88						0.88	
11-Feb	12:30	4	0.5	0.07	1.91	1.91	0.96	0.299	0.28	0.26	0.3	0.28	0.02	0.54	0.83						0.83	
13-Feb	13:00	3.5	0.5	0.07	2.07	2.32	1.45	0.155	0.23	0.22	0.22	0.22	0.01	0.81	0.97						0.97	
16-Feb	10:30	2.3	0.5	0.07	1.78	2	1.45	0.089	0.33	0.32	0.28	0.31	0.03	0.67	0.76						0.76	
18-Feb	10:25	2.8	0.5	0.07	2.27	2.01	1.2	0.277	0.28	0.29	0.28	0.28	0.01	0.63	0.90						0.90	
																						reduced flow rate
20-Feb	11:15	4	0.5	0.07	2.78	2.82	1.94	0.156	0.22	0.22	0.2		0.01									through SSF
23-Feb	10:25	4.1	0.5	0.05	3.08	2.93	1.67	0.266	0.37	0.37	0.38	0.37	0.01	0.65	0.92						0.92	
25-Feb	10:31	3.4	0.5	0.06	2.66	2.57	1.6	0.221	0.27	0.29	0.28	0.28	0.01	0.76	0.98						0.98	
1-Mar	12:00	3.5	0.5	0.06	4.58	5.37	3.41	0.128	0.3	0.27	0.28	0.28	0.02	1.08	1.21						1.21	
3-Mar	12:00	4.1	0.5	0.05	27.4	37.6	26.7	0.011	0.42	0.43	0.44	0.43	0.01	1.79	1.80						1.80	
5-Mar	12:00		0.5	0.06	33.8	30.8	20.5	0.217	0.66	0.62	0.66	0.65	0.02	1.50	1.72						1.72	
5-Mar	12:00		0.5	0.06					0.63			0.63										
5-Mar	12:00		0.5	0.06					0.62			0.62										
8-Mar	11:35	4.4	0.25	0.06	48.3	47.1	23.7	0.309	0.54	0.57	0.52	0.54	0.03	1.64	1.95	0.63	0.67	0.8	0.70	-0.11	1.839	
12-Mar	12:40		0.25	0.06	22	17.9	9.93	0.345	0.25	0.23	0.27	0.25	0.02	1.60	1.94						1.94	
15-Mar	11:30	5.3	0.25	0.05	15.8	15.1	12.9	0.088	0.22	0.21	0.22	0.22	0.01	1.77	1.86						1.86	
17-Mar	11:10	2.9	0.5	0.05	7.26	6.91	5.82	0.096	0.39	0.37	0.41	0.39	0.02	1.17	1.27						1.27	
22-Mar	11:35	2.9	0.5	0.03	20.2	17.9	15.9	0.104	0.64	0.62	0.68	0.65	0.03	1.39	1.49	0.98	0.8	0.72	0.83	-0.11	1.385	
26-Mar	17:00		0.5	0.06	44.4	48.5	30.9	0.157	1.5			1.50		1.31	1.47	1.45				0.01	1.486	
29-Mar	10:35	8.4	0.5	0.06	41.6	36.5	35.8	0.065	1.2	1.17	1.18	1.18	0.02	1.48	1.55	1.1	1.18	1.17	1.15	0.01	1.558	
31-Mar	13:25	8.3	0.25	0.04	47.5	43.5	39.5	0.080	1.08	1.12	1.1	1.10	0.02	1.56	1.64	0.95	0.94	0.95	0.95	0.07	1.700	
5-Apr	11:50	6.5	0.25	0.07	12.6	11.18	6.96	0.258	0.31	0.33	0.32	0.32	0.01	1.34	1.60	0.27	0.23	0.23	0.24	0.12	1.714	
7-Apr	11:05	7.5	0.25	0.05	5.8	7.04	4.22	0.138	0.25	0.26	0.26	0.26	0.01	1.22	1.35	0.18	0.18	0.19	0.18	0.15	1.500	
12-Apr	11:05	9.5	0.5	0.05	5.72	5.98	2.92	0.292	0.15	0.16	0.16	0.16	0.01	1.27	1.56	0.15	0.16	0.17	0.16	-0.01	1.553	
14-Apr	11:55	9	0.5	0.05	3.88	3.37	1.78	0.338	0.16	0.14	0.15	0.15	0.01	1.07	1.41	0.12	0.13	0.14	0.13	0.06	1.475	
19-Apr	13:20	15.3	0.5	0.04	29.9	29.8	18.9	0.199	0.16	0.16	0.16	0.16	0.00	2.07	2.27	0.21	0.24	0.21	0.22	-0.14	2.133	
																						reduced flow rate
21-Apr	11:20		0.5	0.04	9.09	7.23	3.98	0.359	0.16	0.16	0.18		0.01			0.17	0.18	0.16				through SSF
26-Apr	13:20	11.2	0.5	0.05	3.34	3.09	1.64	0.309	0.15	0.14	0.12	0.14	0.02	1.08	1.39	0.17	0.19	0.2	0.19	-0.14	1.253	
																						reduced flow rate
7-May	11:30	14.7		0.05	5.52	4.09	2.34	0.373	0.16	0.15	0.15	0.15	0.01	1.18	1.56						1.56	through SSF (T2)
10-May	11:45	14.6	0.5	0.06	17.8	15.7	7.56	0.372	0.16	0.16	0.17	0.16	0.01	1.67	2.04						2.04	
17-May	10:57	17.4	0.5	0.07	5.32	4.8	1.19	0.650	0.12	0.1	0.11	0.11	0.01	1.03	1.68						1.68	
27-May	11:15	17.4	0.5	0.07	21.4	13.2	5.33	0.604	0.16	0.19	0.2	0.18	0.02	1.46	2.07						2.07	
31-May	13:45	17.2	0.5	0.07	6.17	5.46			0.15	0.16	0.15	0.15	0.01		1.60						1.60	

			overall						
Turbidity - NTU			average	15.20	10.07	0.24	0.40	1.25	1.49
B - blank			stdev	15.10	11.45	0.15	0.34	0.40	0.40
0 - source water									
1 - roughing filter influent	Feb11 to Mar 17 (cold water)								
2 - slow sand filter influent			average	12.55	8.09	0.19	0.38	1.12	1.32
3 - GAC influent			stdev	14.86	9.22	0.10	0.15	0.48	0.47
4 - effluent			n	14	14	14	15	13	13
i/ii/iii/iv - sampling ports	Apr 19 to May 31 (warm water)								
			average	12.32	5.85	0.41	0.15	1.42	1.80
			stdev	9.60	6.17	0.16	0.02	0.40	0.32
			n	8	7	7	7	6	7

Table C.4: Manual Handheld Turbidity Data – Pilot 1 (Train 2)

								Train 2												
Date	**Time**	**Temp (°C)**	**Flow**	**Turb. (B)**	**Turb. (0)**	**Turb. (1)**	**Turb. (2)**	**Log rem. (2)**	**Turb. (3)**			**ave.**	**Log rem. (3)**	**Cum. Log rem (3)**	**Turb. (4)**				**ave.**	**Log rem. (4)**	**cum. Log rem. (4)**	**comments**
2-Feb	18:00		0.3	0.05	3.12	3.84	1.19	0.419	0.25	0.17	0.16	0.19	0.79	1.21						1.21	
9-Feb	14:00	7.3	0.3	0.08	2.42	2.42	0.91	0.425	0.21	0.35	0.27	0.28	0.52	0.94						0.94	
11-Feb	12:30	4	0.5	0.07	1.9	1.88	1.61	0.072	0.19	0.18	0.19	0.19	0.94	1.01						1.01	
13-Feb	13:00	3.5	0.5	0.07	2.18	2.34	2.12	0.012	0.21	0.18	0.17	0.19	1.06	1.07						1.07	
16-Feb	10:30	2.3	0.5	0.07	1.92	1.81	1.25	0.186	0.25	0.26	0.22	0.24	0.71	0.90						0.90	
18-Feb	10:25	2.8	0.5	0.07	2.22	2.91	1.77	0.098	0.29	0.29	0.28	0.29	0.79	0.89						0.89	
																					reduced flow rate
20-Feb	11:15	4	0.5	0.07	2.83	2.77	1.55	0.261	0.18	0.17	0.18										through SSF
23-Feb	10:25	4.1	0.5	0.04	2.96	3.09	1.56	0.278	0.37	0.38	0.37	0.37	0.62	0.90						0.90	
25-Feb	10:31	3.4	0.5	0.06	2.82	2.58	1.41	0.301	0.33	0.35	0.35	0.34	0.61	0.91	0.29	0.28	0.35	0.31	0.049	0.964	
1-Mar	12:00	3.5	0.5	0.06	4.88	4.08	1.65	0.471	0.26	0.22	0.22	0.23	0.85	1.32	0.2	0.21	0.21	0.21	0.053	1.373	
3-Mar	12:00	4.1	0.5	0.05	30.2	26.4	28.1	0.031	0.25	0.25	0.26	0.25	2.05	2.08	0.3	0.37	0.28	0.32	-0.097	1.979	
5-Mar	12:00		0.5	0.07	33.8	30.5	14	0.383	0.58	0.51	0.58	0.56	1.40	1.78	0.46	0.49	0.47	0.47	0.070	1.854	
5-Mar	12:00																				
5-Mar	12:00																				
8-Mar	11:35	4.4	0.25	0.06	55.5	44.4	19.6	0.452	0.44	0.45	0.43	0.44	1.65	2.10						2.10	
12-Mar	12:40		0.25	0.07	21.2	16.7	5.57	0.580	0.35	0.25	0.29	0.30	1.27	1.85						1.85	
15-Mar	11:30	5.3	0.25	0.05	14.7	15.1	6.77	0.337	0.3	0.28	0.28	0.29	1.37	1.71						1.71	
17-Mar	11:10	2.9	0.5	0.05	6.66	7.82	6.48	0.012	0.61	0.6	0.64	0.62	1.02	1.03						1.03	
22-Mar	11:35	2.9	0.5	0.03	21.4	18.8	13.2	0.210	1.19	1.25	1.22	1.22	1.03	1.24	1.16	1.2	1.12	1.16	0.022	1.266	
26-Mar	17:00		0.5	0.06	44.4	52.3	30.4	0.165	1.44			1.44	1.32	1.49	1.39			1.39	0.015	1.504	
29-Mar	10:35	8.4	0.5	0.06	47.4	38.2	29.4	0.207	1.48	1.5	1.52	1.50	1.29	1.50	1.41	1.43	1.43	1.42	0.023	1.522	
31-Mar	13:25	8.3	0.25	0.05	56.5	52.1	24.4	0.365	0.83	0.85	0.83	0.84	1.46	1.83	0.62	0.63	0.65	0.63	0.121	1.950	
5-Apr	11:50	6.5	0.25	0.07	11.8	11.4	3.19	0.568	0.39	0.41	0.39	0.40	0.91	1.47	0.29	0.27	0.3	0.29	0.141	1.615	
7-Apr	11:05	7.5	0.25	0.05	6.38	7.46	1.88	0.531	0.32	0.31	0.3	0.31	0.78	1.31	0.19	0.21	0.2	0.20	0.190	1.504	
12-Apr	11:05	9.5	0.5	0.04	4.53	5.38	4.6	-0.007	0.19	0.22	0.23	0.21	1.33	1.33	0.18	0.19	0.21	0.19	0.043	1.370	
14-Apr	11:55	9	0.5	0.05	5.91	4.02	2.29	0.412	0.2	0.21	0.2	0.20	1.05	1.46	0.19	0.19	0.18	0.19	0.037	1.501	
19-Apr	13:20	15.3	0.75	0.05	29.5	27.7	21.9	0.129	0.38	0.37	0.38	0.38	1.76	1.89	0.37	0.39	0.36	0.37	0.004	1.898	
																					reduced flow rate
21-Apr	11:20		0.75	0.04	8.7	7.29	6.01	0.161	0.18	0.16	0.18				0.19	0.18	0.19				through SSF
26-Apr	13:20	11.2	1	0.05	5.3	2.99	1.9	0.446	0.23	0.25	0.25	0.24	0.89	1.34	0.36	0.41	0.4	0.39	-0.205	1.133	
																					reduced flow rate
7-May	11:30	14.7	1	0.05	5.52	4.19	3.09	0.252	0.24	0.23	0.22										through SSF (T2)
10-May	11:45	14.6	1	0.06	16.9	15.3	10	0.228	0.48	0.46	0.47	0.47	1.33	1.56						1.56	
17-May	10:57	17.4	1	0.07	5.47	6.28	2.64	0.316	0.21	0.21	0.21	0.21	1.10	1.42						1.42	
27-May	11:15	17.4	0.5	0.07	20.2	10.8	3.5	0.761	0.14	0.15	0.13	0.14	1.40	2.16						2.16	
31-May	13:45	17.2	0.5	0.07	5.77	5.05	1.97	0.467	0.11	0.11	0.11	0.11	1.25	1.72						1.72	

		overall						
Turbidity - NTU		average	15.98	8.46	0.29	0.44	1.16	1.45
B - blank		stdev	16.73	9.49	0.19	0.38	0.35	0.39
0 - source water								
1 - roughing filter influent	Feb11 to Mar 17 (cold water)							
2 - slow sand filter influent		average	13.13	6.67	0.25	0.33	1.10	1.35
3 - GAC influent		stdev	16.40	8.26	0.19	0.13	0.43	0.48
4 - effluent		n	14	14	14	13	13	13
i/ii/iii/iv - sampling ports	Apr 19 to May 31 (warm water)							
		average	12.17	6.38	0.34	0.26	1.29	1.68
		stdev	9.08	6.83	0.21	0.14	0.29	0.31
		n	8	8	8	6	6	6

Table C.5: Manual Handheld Turbidity Data – Pilot 2

Date	Time	Temp. (°C)	Flow (Lpm)	Turb. (B)	Turb. (1)	Turb. (2A)	log rem. (2A)	Turb. (2B)	log rem. (2B)	turb. difference (2A and 2B)	log rem. difference (2B and 2A)	Turb. (3)			ave.	stdev	log rem. (3)	Turb. (4)			ave.	log rem. (4)	cum. Log rem.
2-Feb	16:00		0.36	0.05	1.57	1.09	0.158	1.13	0.143	0.040	-0.016	0.21	0.22	0.28	0.24	0.038	0.663	0.27	0.24	0.3	0.27	-0.057	0.765
9-Feb	14:00	7.3	0.36	0.08	1.55	1.42	0.038	1.01	0.186	-0.410	0.148	0.29	0.35	0.2	0.28	0.075	0.705	0.16	0.18	0.21	0.18	0.184	0.927
11-Feb	11:35	4	0.36	0.07	2.82	0.88	0.506	0.76	0.569	-0.120	0.064	0.33	0.27	0.43	0.34	0.081	0.409	0.23	0.22	0.26	0.24	0.162	1.076
13-Feb	10:25	3.5	0.36	0.07	2.1	1.09	0.285	0.82	0.408	-0.270	0.124	0.24	0.24	0.24	0.24	0.000	0.657	0.27	0.26	0.27	0.27	-0.046	0.896
16-Feb	11:25	2.3	0.36	0.05	2.01	1.29	0.193	1.13	0.250	-0.160	0.058	0.23	0.24	0.24	0.24	0.006	0.736	0.32	0.28	0.28	0.29	-0.093	0.836
18-Feb	12:20	2.8	0.36	0.06	2.01	1.32	0.183	1.08	0.270	-0.240	0.087	0.24	0.25	0.25	0.25	0.006	0.728	0.26	0.26	0.27	0.26	-0.028	0.883
20-Feb	12:25	3.7	0.36	0.05	2.27	1.54	0.169	1.19	0.280	-0.350	0.112	0.3	0.29	0.29	0.29	0.006	0.720	0.29	0.29	0.29	0.29	0.005	0.894
23-Feb	11:45	3.8	0.48	0.05	2.74	2.04	0.128	1.52	0.256	-0.520	0.128	0.45	0.45	0.44	0.45	0.006	0.660	0.34	0.36	0.36	0.35	0.102	0.890
25-Feb	13:20	5	0.48	0.06	2.36	1.8	0.118	1.24	0.279	-0.560	0.162	0.37	0.36	0.36	0.36	0.006	0.695	0.34	0.34	0.32	0.33	0.037	0.850
1-Mar	12:45	4.2	0.48	0.05	4.14	2.46	0.226	1.31	0.500	-1.150	0.274	0.25	0.26	0.23	0.25	0.015	0.999	0.21	0.21	0.22	0.21	0.063	1.288
3-Mar	12:20	5.2	0.48	0.05	22	10.1	0.338	3.89	0.752	-6.210	0.414	0.25	0.25	0.25	0.25	0.000	1.606	0.22	0.23	0.23	0.23	0.043	1.987
5-Mar	12:00		0.48	0.05	43.1			9.68	0.649			0.3	0.31	0.3	0.30	0.006		0.24	0.26	0.26	0.25	0.078	2.231
8-Mar	12:45	5	0.48	0.06	44	26.8	0.215	15.5	0.453	-11.300	0.238	0.58	0.59	0.63	0.60	0.026	1.650	0.28	0.29	0.31	0.29	0.311	2.176
12-Mar	13:15		0.24	0.07	16.1	7.81	0.314	4.4	0.563	-3.410	0.249	0.45	0.43	0.43	0.44	0.012	1.253	0.25	0.31	0.27	0.28	0.198	1.765
15-Mar	11:45	5.9	0.48	0.04	9.43	6.73	0.146	4.14	0.358	-2.590	0.211	0.39	0.41	0.4	0.40	0.010	1.226	0.26	0.25	0.26	0.26	0.193	1.565
17-Mar	13:23	4.3	0.48	0.02	6.25	4.71	0.123	3.33	0.273	-1.380	0.151	0.4	0.41	0.44	0.42	0.021	1.053	0.24	0.27	0.26	0.26	0.210	1.387
22-Mar	10:55	4.8	0.72	0.06	17.3	12.9	0.127	8.24	0.322	-4.660	0.195	1.79	1.89	1.85	1.84	0.050	0.845	1.01	1.06	1.05	1.04	0.249	1.221
24-Mar	10:45	5.5	0.72	0.05	6.37	5.21	0.087	2.72	0.370	-2.490	0.282	0.58	0.55	0.56	0.56	0.015	0.966	0.38	0.37	0.37	0.37	0.179	1.232
26-Mar	17:00		0.72	0.05	56.3	42.2	0.125	13.1	0.633	-29.100	0.508	1.27			1.27		1.522	0.59			0.59	0.333	1.980
29-Mar	11:45	9.4	0.72	0.06	29.3	18.6	0.197	11	0.425	-7.600	0.228	0.74	0.77	0.78	0.76	0.021	1.387	0.39	0.38	0.38	0.38	0.299	1.883
31-Mar	11:50	9	0.72	0.05	23.1	15.7	0.168	6.41	0.557	-9.290	0.389	0.98	0.98	0.96	0.97	0.012	0.819	0.59	0.56	0.6	0.58	0.222	1.598
5-Apr	12:00	6.6	0.72	0.04	11.1	7.29	0.183	3.85	0.460	-3.440	0.277	0.69	0.72	0.71	0.71	0.015	0.736	0.32	0.33	0.35	0.33	0.326	1.522
7-Apr	10:45	6.7	0.72	0.04	11	3.54	0.492	2.25	0.689	-1.290	0.197	0.53	0.56	0.54	0.54	0.015	0.617	0.3	0.32	0.31	0.31	0.244	1.550
12-Apr	12:10	8.9	0.48	0.04	4.17	1.69	0.392	1.07	0.591	-0.620	0.199	0.27	0.3	0.27	0.28	0.017	0.582	0.16	0.17	0.16	0.16	0.234	1.407
14-Apr	10:40	8.2	0.48	0.05	2.63	1.41	0.271	0.97	0.433	-0.440	0.162	0.26	0.24	0.27	0.26	0.015	0.577	0.18	0.18	0.18	0.18	0.154	1.165
19-Apr	12:25	13.3	0.48	0.04	22.8	12.1	0.275	6.06	0.575	-6.040	0.300	0.27	0.3	0.31	0.29	0.021	1.315	0.19	0.21	0.23	0.21	0.145	2.036
26-Apr	14:27	11.8	0.48	0.05	2.95	1.75	0.227	0.54	0.737	-1.210	0.511	0.26	0.25	0.27	0.26	0.010	0.317	0.17	0.2	0.19	0.19	0.144	1.199
28-Apr	11:00	10	0.48	0.04	2.09	1.55	0.130	0.551	0.579	-0.999	0.449	0.26	0.26	0.26	0.26	0.000	0.326	0.22	0.2	0.2	0.21	0.100	1.005
3-May	11:10	10.9	0.48	0.05	8.01	3.28	0.388	1.05	0.882	-2.230	0.495	0.24	0.25	0.25	0.25	0.006	0.629	0.19	0.2	0.19	0.19	0.106	1.617
10-May	11:00	13	0.48	0.05	11.6	4.52	0.409	2.29	0.705	-2.230	0.295	0.24	0.26	0.25	0.25	0.010	0.962	0.2	0.2	0.21	0.20	0.090	1.756

				overall										overall									
Turbidity - NTU				average	12.81	7.20	0.23	3.83	0.48	-3.58	0.25			average	0.47		0.88				0.31	0.15	1.41
B - blank				stdev	14.47	9.31	0.12	4.04	0.18	5.81	0.13			stdev	0.36		0.37				0.17	0.11	0.44
1 - roughing filter influent										28	28			n	29						29		
2 - slow sand filter 1 influent			Feb 9 to Mar 17 (cold water)											Feb 9 to Mar 17 (cold water)									
3 - slow sand filter 2 influent				average	10.86	5.00	0.21	3.40	0.40					average	0.34		0.94				0.27	0.09	1.31
4 - effluent														max	0.60		1.65				0.35	0.31	2.23
i/ii/iii/iv - sampling ports														min	0.24		0.41				0.18	-0.09	0.84
				stdev	14.53	6.92	0.12	4.10	0.17					stdev	0.10		0.38				0.04	0.11	0.51
				n	15	14	14	15	15					n	15		14				15	15	15
			Apr 19 to May 10 (warm water)											Apr 19 to May 10 (warm water)									
				average	9.49	4.64	0.29	2.10	0.70					average	0.26		0.71				0.20	0.12	1.52
							0.41		0.88					max	0.29		1.32				0.21	0.15	2.04
							0.13		0.58					min	0.25		0.32				0.19	0.09	1.00
				stdev	8.39	4.34	0.12	2.33	0.13					stdev	0.02		0.43				0.01	0.03	0.42
				n	5	5	5	5	5					n	5		5				5	5	5

Table C.6: Paired T-test: Determination of the Effect of Increased Hydraulic Loading Rate on Effluent Turbidity

		Train 1	Train 2	
Date	Time	Turb. (3)	Turb. (3)	Difference
26-Apr	13:58	0.14	0.25	0.11
7-May	12:00	0.15	0.22	0.07
10-May	12:25	0.16	0.47	0.31
17-May	11:30	0.11	0.21	0.10
			mean	0.15
			stdev	0.108
			null hypothesis	u_1-u_2=0
			t calc (=mean/stdev*sqrt(n))	2.705
			t stat (n-1, 0.025)	3.182
			reject null hypothesis?	no

Conclusion: not enough sample data (insufficient degrees of freedom)

Table C.7: Statistical T-test Analyses – Pilot 1

Test objective: Compare influent turbidity in warm vs. cold water (Pilot system 1)

F-test to determine is sample variances are equal:

s_1 (larger)=	14.86	v_1=	14	u_1=	12.55
s_2 (smaller)=	9.6	v_2=	8	u_2=	12.32
s_1^2=	220.820				
s_2^2=	92.160				
Fcalc=	2.40				
Fcrit (0.05, v1, v2)=	3.22				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	13.259
tcalc=	0.039
tcrit (0.025, v_1+v_2-2)=	2.086
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	no

Test objective: Compare effluent turbidity in warm vs. cold water (Pilot system 1-Train 1)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.15	v_1=	15	u_1=	0.38
s_2 (smaller)=	0.02	v_2=	7	u_2=	0.15
s_1^2=	0.023				
s_2^2=	0.0004				
Fcalc=	56.25				
Fcrit (0.05, v1, v2)=	3.51				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	yes				

T-test to compare two samples:

df=	15.036
tcalc=	5.829
tcrit (0.025, df)=	2.131
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	yes

Test objective: Compare effluent turbidity in warm vs. cold water (Pilot system 1-Train 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.14	v_1=	6	u_1=	0.26
s_2 (smaller)=	0.13	v_2=	13	u_2=	0.33
s_1^2=	0.020				
s_2^2=	0.017				
Fcalc=	1.16				
Fcrit (0.05, v1, v2)=	2.92				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	0.133
tcalc=	-1.066
tcrit (0.025, v_1+v_2-2)=	2.11
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	no

Test objective: Compare roughing filter log removals for warm vs. cold water (Pilot system 1-train 1)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.16	v_1=	7	u_1=	0.41
s_2 (smaller)=	0.1	v_2=	14	u_2=	0.19
s_1^2=	0.026				
s_2^2=	0.010				
Fcalc=	2.56				
Fcrit (0.05, v1, v2)=	2.76				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	0.122
tcalc=	3.890
tcrit (0.025, v_1+v_2-2)=	2.093
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	yes

Test objective: Compare roughing filter log removals for warm vs. cold water (Pilot system 1-train 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.21	v_1=	8	u_1=	0.34
s_2 (smaller)=	0.19	v_2=	14	u_2=	0.25
s_1^2=	0.044				
s_2^2=	0.036				
Fcalc=	1.22				
Fcrit (0.05, v1, v2)=	2.7				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	0.197
tcalc=	1.030
tcrit (0.025, v_1+v_2-2)=	2.086
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	no

Table C.8: Statistical T-test Analyses – Pilot 1 (cont'd)

Test objective: Compare slow sand filter log removals for warm vs. cold water (Pilot system 1-train 1)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.48	v_1=	13	u_1=	1.12
s_2 (smaller)=	0.4	v_2=	6	u_2=	1.42
s_1^2=	0.230				
s_2^2=	0.160				
Fcalc=	1.44				
Fcrit (0.05, v1, v2)=	3.99				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	0.458
tcalc=	-1.327
tcrit (0.025, v_1+v_2-2)=	2.11
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	no

Test objective: Compare slow sand filter log removals for warm vs. cold water (Pilot system 1-train 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.43	v_1=	13	u_1=	1.1
s_2 (smaller)=	0.29	v_2=	6	u_2=	1.29
s_1^2=	0.185				
s_2^2=	0.084				
Fcalc=	2.20				
Fcrit (0.05, v1, v2)=	3.99				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	0.394
tcalc=	-0.977
tcrit (0.025, v_1+v_2-2)=	2.11
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	no

Test objective: Compare cumulative log removals for warm vs. cold water (Pilot system 1-train 1)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.47	v_1=	13	u_1=	1.32
s_2 (smaller)=	0.32	v_2=	7	u_2=	1.8
s_1^2=	0.221				
s_2^2=	0.102				
Fcalc=	2.16				
Fcrit (0.05, v1, v2)=	3.55				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	0.426
tcalc=	-2.404
tcrit (0.025, v_1+v_2-2)=	2.101
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	yes

Test objective: Compare cumulative log removals for warm vs. cold water (Pilot system 1-train 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.48	v_1=	13	u_1=	1.68
s_2 (smaller)=	0.31	v_2=	6	u_2=	1.35
s_1^2=	0.230				
s_2^2=	0.096				
Fcalc=	2.40				
Fcrit (0.05, v1, v2)=	3.98				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	0.437
tcalc=	1.530
tcrit (0.025, v_1+v_2-2)=	2.11
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	no

Table C.9: Statistical T-test Analyses – Pilot 2

Test objective: Compare influent turbidity in warm vs. cold water (Pilot system 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	14.53	v_1=	15	u_1=	10.86
s_2 (smaller)=	8.39	v_2=	5	u_2=	9.49
s_1^2=	211.121				
s_2^2=	70.392				
Fcalc=	3.00				
Fcrit (0.05, v1, v2)=	4.62				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	13.411
tcalc=	0.198
tcrit (0.025, v_1+v_2-2)=	2.101
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	no

Test objective: Compare final effluent turbidity in warm vs. cold water (Pilot system 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.04	v_1=	15	u_1=	0.27
s_2 (smaller)=	0.01	v_2=	5	u_2=	0.2
s_1^2=	0.002				
s_2^2=	0.0001				
Fcalc=	16.00				
Fcrit (0.05, v1, v2)=	4.62				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	yes				

T-test to compare two samples:

df=	17.579
tcalc=	6.220
tcrit (0.025, df)=	2.101
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	yes

Test objective: Compare slow sand filter effluent turbidity in warm vs. cold water (Pilot system 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.1	v_1=	15	u_1=	0.34
s_2 (smaller)=	0.02	v_2=	5	u_2=	0.26
s_1^2=	0.010				
s_2^2=	0.000				
Fcalc=	25.00				
Fcrit (0.05, v1, v2)=	4.62				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	yes				

T-test to compare two samples:

df=	16.719
tcalc=	2.928
tcrit (0.025, df)=	2.11
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	yes

Test objective: Compare roughing filter A log removals for warm vs. cold water (Pilot system 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.12	v_1=	14	u_1=	0.21
s_2 (smaller)=	0.12	v_2=	5	u_2=	0.29
s_1^2=	0.014				
s_2^2=	0.014				
Fcalc=	1.00				
Fcrit (0.05, v1, v2)=	4.64				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	0.120
tcalc=	-1.280
tcrit (0.025, v_1+v_2-2)=	2.11
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	no

Table C.10: Statistical T-test Analyses – Pilot 2 (cont'd)

Test objective: Compare roughing filter B log removals for warm vs. cold water (Pilot system 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.17	v_1=	15	u_1=	0.4
s_2 (smaller)=	0.13	v_2=	5	u_2=	0.7
s_1^2=	0.029				
s_2^2=	0.017				
Fcalc=	1.71				
Fcrit (0.05, v1, v2)=	4.62				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	0.162
tcalc=	-3.587
tcrit (0.025, v_1+v_2-2)=	2.101
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	yes

Test objective: Compare slow sand filter 1 log removals for warm vs. cold water (Pilot system 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.43	v_1=	5	u_1=	0.71
s_2 (smaller)=	0.38	v_2=	14	u_2=	0.94
s_1^2=	0.185				
s_2^2=	0.144				
Fcalc=	1.28				
Fcrit (0.05, v1, v2)=	4.64				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	0.392
tcalc=	-1.125
tcrit (0.025, v_1+v_2-2)=	2.11
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	no

Test objective: Compare slow sand filter 2 log removals for warm vs. cold water (Pilot system 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.11	v_1=	15	u_1=	0.09
s_2 (smaller)=	0.03	v_2=	5	u_2=	0.12
s_1^2=	0.012				
s_2^2=	0.001				
Fcalc=	13.44				
Fcrit (0.05, v1, v2)=	4.62				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	yes				

T-test to compare two samples:

df=	17.837
tcalc=	-0.955
tcrit (0.025, df)=	2.101
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	no

Test objective: Compare cumulative log removals for warm vs. cold water (Pilot system 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.51	v_1=	15	u_1=	1.31
s_2 (smaller)=	0.42	v_2=	5	u_2=	1.52
s_1^2=	0.260				
s_2^2=	0.176				
Fcalc=	1.47				
Fcrit (0.05, v1, v2)=	4.62				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	no				

T-test to compare two samples:

s_p=	0.491
tcalc=	-0.828
tcrit (0.025, v_1+v_2-2)=	2.101
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	no

Test objective: Compare effluent turbidity from slow sand filter 1 and 2 (Pilot system 2)

F-test to determine is sample variances are equal:

s_1 (larger)=	0.36	v_1=	29	u_1=	0.47
s_2 (smaller)=	0.17	v_2=	29	u_2=	0.31
s_1^2=	0.130				
s_2^2=	0.029				
Fcalc=	4.48				
Fcrit (0.05, v1, v2)=	1.86				
Null hypothesis=	$\sigma_1=\sigma_2$				
Reject null? (Fcalc>Fcrit)	yes				

T-test to compare two samples:

df=	39.896
tcalc=	2.164
tcrit (0.025, df)=	2.021
Null hypothesis=	$u_1=u_2$
Reject null? (tcalc>tcrit)	yes

Table C.11: Paired T-test: Determination of the Effect of Ozonation on Effluent Turbidity

Date	Time	Train 1 Turb. (3)	Train 2 Turb. (3)	Difference
25-Feb	10:31	0.28	0.34	0.06
1-Mar	12:00	0.28	0.23	-0.05
3-Mar	12:00	0.43	0.25	-0.18
5-Mar	12:00	0.65	0.56	-0.09
8-Mar	11:35	0.54	0.00	-0.54
12-Mar	12:40	0.25	0.00	-0.25
15-Mar	11:30	0.22	0.44	0.22
17-Mar	11:10	0.39	0.30	-0.09
22-Mar	11:35	0.65	0.29	-0.36
			mean	-0.14
			stdev	0.227
			null hypothesis	u_1-u_2=0
		t calc (=mean/stdev*sqrt(n))		-1.878
			t stat (n-1, 0.025)	2.306
			reject null hypothesis?	no

Table C.12: Paired T-test: Determination of Difference in Effluent Turbidity from Roughing Filter A and B (Pilot 2)

mean of difference	-3.58
stdev of difference	5.81
null hypothesis	u_1-u_2=0
t calc (=mean/stdev*sqrt(n))	-3.263
t stat (n-1, 0.025)	2.052
reject null hypothesis?	yes

Table C.13: Paired T-test: Determination of Difference in Log Removals from Roughing Filter A and B (Pilot 2)

mean of difference	0.25
stdev of difference	0.13
null hypothesis	u_1-u_2=0
t calc (=mean/stdev*sqrt(n))	9.770
t stat (n-1, 0.025)	2.052
reject null hypothesis?	yes

Table C.14: Paired T-test: Determination of Difference in Effluent Turbidity and Log Removals from Roughing Filters in Pilot System 1 and Pilot System 2 (Roughing Filter A)

Pilot System 1							Pilot System 2					Difference			
			Train 1		Train 2										
Date	Time	Flow	Turb. (2)	Log rem. (2)	Turb. (2)	Log rem. (2)	Date	Time	Flow	Turb. (2A)	log rem. (2A)	Turb. difference compared to Train1	log rem. difference compared to Train1	Turb. difference compared to Train2	log rem. difference compared to Train2
23-Feb	10:25	0.5	1.67	0.266	1.56	0.278	23-Feb	11:45	0.5	2.04	0.128	0.37	-0.138	0.48	-0.150
1-Mar	12:00	0.5	3.41	0.128	1.65	0.471	1-Mar	12:45	0.5	2.46	0.226	-0.95	0.098	0.81	-0.245
3-Mar	12:00	0.5	26.7	0.011	28.10	0.031	3-Mar	12:20	0.5	10.1	0.338	-16.6	0.327	-18.00	0.307
12-Mar	12:40	0.25	9.93	0.345	5.57	0.580	12-Mar	13:15	0.2	7.81	0.314	-2.12	-0.031	2.24	-0.266
17-Mar	11:10	0.5	5.82	0.096	6.48	0.012	17-Mar	13:23	0.5	4.71	0.123	-1.11	0.027	-1.77	0.111
12-Apr	11:05	0.5	2.92	0.292	4.60	-0.007	12-Apr	12:10	0.5	1.69	0.392	-1.23	0.100	-2.91	0.399
14-Apr	11:55	0.5	1.78	0.338	2.29	0.412	14-Apr	10:40	0.5	1.41	0.271	-0.37	-0.068	-0.88	-0.141
19-Apr	13:20	0.5	18.9	0.199	21.90	0.129	19-Apr	12:25	0.5	12.1	0.275	-6.8	0.076	-9.80	0.146
26-Apr	13:20	0.5	1.64	0.309	1.90	0.446	26-Apr	14:27	0.5	1.75	0.227	0.11	-0.082	-0.15	-0.219
		average	8.086	0.221	8.228	0.261			average	4.897	0.255	-3.189	0.034	-3.331	-0.007
		stdev	8.965	0.119	9.799	0.226			stdev	4.092	0.090	5.462	0.139	6.505	0.252
		n	9	9	9	9			n	9	9	9	9	9	9
											null hypothesis	u_1-u_2=0	u_1-u_2=0	u_1-u_2=0	u_1-u_2=0
											t calc (=mean/stdev*sqrt(n))	-1.751	0.743	-1.536	-0.077
											t stat (n-1, 0.025)	2.306	2.306	2.306	2.306
Note: dates with similar operational conditions were chosen for comparing pilot system 1 and 2											reject null hypothesis?	no	no	no	no

Appendix D: Total and Fecal Coliform Data and Analysis

Table D.1: Total and Fecal Coliform Data – Pilot 1

Pilot System 1 - Train 1

				Raw water (0)					Ozone Contactor Effluent (1)								Roughing Filter effluent (2)										Slow Sand Filter 1 effluent (3)									
				Total coliforms (MPN/100mL)		Fecal coliforms (FC/100mL)			Total coliforms (MPN/100mL)				Fecal coliforms (FC/100mL)				Total coliforms (MPN/100mL)						Fecal coliforms (FC/100mL)					Total coliforms (MPN/100mL)						Fecal coliforms (FC/100mL)				
Date	Time	Temp.	Raw Turb.	Pres.	Conf.	Pres.	Conf.	Compl.	Pres.	Conf.	Graph value	log rem.	Pres.	Conf.	Compl.	log rem.	Pres.	Conf.	Graph value	log rem.	cum. Log rem.	Pres.	Conf.	Compl.	log rem.	cum. Log rem.	Pres.	Conf.	Graph value	log rem.	cum. log rem.	Pres.	Conf.	Compl	log rem.	cum. log rem.
02/23/04	13:30	4.1	3.02	1300	800	520	470	360	1700	1300		-0.211	130	117	104	0.539	500	220		0.772	0.561	54	49	49	0.327	0.866	30	17		1.112	1.673	10	8	8	0.787	1.653
03/01/04	11:30	3.5	4.88	600	600	120	108	108	2		1	2.477	0.1			3.033	2	2		0.000	2.477	0.1			0.000	3.033	2	2		0.000	2.477	1	0.1	0.1	0.000	3.033
03/08/04	11:30	4.4	19.71	2400	2400	380	342	304																			2		1		3.079	0.1				3.483
03/15/04	10:45	5.3	12.08	800	800	176	176	158	2		1	2.602					2		1	0.000	2.602						2		1	0.000	2.602	0.1				3.199
03/22/04	12:55	2.9	19.64	240	240	112	101	90																			2		1		2.079	0.1				2.954
03/29/04	11:30	8.4	35.82	800	800	240	240	240	2		1	2.602					2		1	0.000	2.602						2		1	0.000	2.602	0.1				3.380
04/05/04	12:30	6.5	11.22	500	500	102	102	92																			2		1		2.398	0.1				2.964
04/12/04	12:05	9.5	4.13	170	170	24	22	22	2		1	1.929					2		1	0.000	1.929						2		1	0.000	1.929	0.1				2.342
04/19/04	14:30	15.3	26.45	1700	1700	2100	1890	1680																			2		1		2.929	0.1				4.225
04/26/04	10:45	11.2	2.92	300	300	54	54	49	2		1	2.176					2		1	0.000	2.176						2		1	0.000	2.176	0.1				2.690
05/10/04	12:30	14.6	14.82	800	800	550	440	440																			2		1		2.602	0.1				3.643
05/17/04	11:00	17.4	4.05	700	700	160	160	160	2			2.544					2			0.000	2.544						2		1	0.000	2.544	0.1				3.204
05/31/04	10:30	17.2	3.41	300	300	290	290	290																			2		1		2.176	0.1				3.462

Pilot System 1 - Train 2

				Raw water (0)					Ozone Contactor Effluent (1)								Roughing Filter effluent (2)										Slow Sand Filter 1 effluent (3)									
				Total coliforms (MPN/100mL)		Fecal coliforms (FC/100mL)			Total coliforms (MPN/100mL)				Fecal coliforms (FC/100mL)				Total coliforms (MPN/100mL)						Fecal coliforms (FC/100mL)					Total coliforms (MPN/100mL)						Fecal coliforms (FC/100mL)				
Date	Time	Temp.	Raw Turb.	Pres.	Conf.	Pres.	Conf.	Compl.	Pres.	Conf.	Graph value	log rem.	Pres.	Conf.	Compl.	log rem.	Pres.	Conf.	Graph value	log rem.	cum. Log rem.	Pres.	Conf.	Compl.	log rem.	cum. Log rem.	Pres.	Conf.	Graph value	log rem.	cum. log rem.	Pres.	Conf.	Compl	log rem.	cum. log rem.
02/23/04	13:45	4.1	3.02	2400	1300	50	40	30	800	800		0.211	160	144	144	-0.681	300	300		0.426	0.637	28	28	25	0.760	0.079	130	34		0.946	1.582	6	4	4	0.796	0.875
03/01/04	11:30	3.5	4.88	220	220	90	90	81	230	230		-0.019	94	75	47	0.236	140	140		0.216	0.196	22	15	11	0.631	0.867	4	2		1.845	2.041	2	2	2	0.740	1.607
03/08/04	11:30	4.4	19.71	2400	2400	380	342	304									130	130			1.266	58	58	52		0.767	7	7		1.269	2.535	2	2	2	1.415	2.182
03/15/04	10:45	5.3	12.08	800	800	176	176	158	700			0.058					110			0.804	0.862						2	2		1.740	2.602	2	2	2		1.898
03/22/04	12:55	2.9	19.64	240	240	112	101	90																			170	170			0.150	40	40	40		0.352
03/29/04	11:30	8.4	35.82	800	800	240	240	240	2		1	2.602					2		1	0.000	2.602						2	2		0.000	2.602	0.1				3.380
04/05/04	12:30	6.5	11.22	500	500	102	102	92																			2		1		2.398	0.1				2.964
04/12/04	12:05	9.5	4.13	170	170	24	22	22	2		1	1.929					2		1	0.000	1.929						2		1	0.000	1.929	0.1				2.342
04/19/04	14:30	15.3	26.45	1700	1700	2100	1890	1680																			2		1		2.929	0.1				4.225
04/26/04	10:45	11.2	2.92	300	300	54	54	49	2		1	2.176					4			-0.301	1.875						2		1	0.301	2.176	0.1				2.690
05/10/04	12:30	14.6	14.82	800	800	550	440	440																			2		1		2.602	0.1				3.643
05/17/04	11:00	17.4	4.05	700	700	160	160	160	2			2.544					2		1	0.000	2.544						2		1	0.000	2.544	0.1				3.204
05/31/04	10:30	17.2	3.41	300	300	290	290	290																			2		1		2.176	0.1				3.462

Legend:

2	= <2 (below detection limit)
1	= dummy number to represent <2 on graph (total coliforms only)
0.1	= 0 (fecal coliforms only)
	= > (greater than) reported removal
	= < (less than) reported removal

Table D.2: Total and Fecal Coliform Data – Pilot 2

				Raw water (1)					Roughing Filter 1 effluent (2A)							Roughing Filter 2 effluent (2B)							
				Total coliforms (MPN/100mL)		Fecal coliforms (FC/100mL)			Total coliforms (MPN/100mL)			Fecal coliforms (FC/100mL)				Total coliforms (MPN/100mL)			Fecal coliforms (FC/100mL)				
Date	Time	Temp.	Raw Turb.	Pres.	Conf.	Pres.	Conf.	Compl.	Pres.	Conf.	log rem.	Pres.	Conf.	Compl.	log rem.	Pres.	Conf.	log rem.	Pres.	Conf.	Compl.	log rem.	
02/23/04	13:00	3.8	3.06	800	800	154	154	108	500	500	0.204	70	56	49	0.343	300	300	0.426	36	36	36	0.477	
03/01/04	12:45	4.2	4.57	170	170	74	59	59	170	170	0.000	16	14	12	0.692	50	50	0.531	10	6	6	0.993	
03/08/04	12:45	5	28.78	2400	2400	510	510	510	1300	340	0.849	490	490	490	0.017	220	140	1.234	164	148	148	0.537	
03/15/04	12:25	5.9	10.30	300	300	62	56	50	110		0.436					80		0.574					
03/22/04	12:40	4.8	19.76	240	240	94	85	75															
03/29/04	12:15	9.4	34.29	500	500	140	140	140	80		0.796					50		1.000					
04/05/04	13:00	6.6	11.14	230	230	28	25	25															
04/12/04	13:00	8.9	3.85	30	30	2	2	2	8		0.574					8		0.574					
04/19/04	13:10	13.3	27.58	2200	2200	1530	1377	1377															
04/26/04	14:52	11.8	2.87	30	30	16	16	8	13		0.363					2		1.176					
05/10/04	11:30	13	15.66	500	500	120	84	84															

				Slow Sand Filter 1 effluent (3)										Slow Sand Filter 2 effluent (4)									
				Total coliforms (MPN/100mL)					Fecal coliforms (FC/100mL)					Total coliforms (MPN/100mL)					Fecal coliforms (FC/100mL)				
Date	Time	Temp.	Raw Turb.	Pres.	Conf.	Graph value	log rem.	cum. log rem.	Pres.	Conf.	Compl	log rem.	cum. log rem.	Pres.	Conf.	Graph value	log rem.	cum. log rem.	Pres.	Conf.	Compl.	log rem.	cum. log rem.
02/23/04	13:00	3.8	3.06	130	130		0.585	0.789	10	8	8	0.787	1.130	30	30		0.637	1.426	6	2	0.1	1.903	3.033
03/01/04	12:45	4.2	4.57	2	2	1	1.929	1.929	1	1	1	1.079	1.771	2	2		0.000	1.929	1	0.1	0.1	1.000	2.771
03/08/04	12:45	5	28.78	2	2		2.230	3.079	2	2	2	2.389	2.407	2		1	0.000	3.079	2	2	2	0.000	2.407
03/15/04	12:25	5.9	10.30	2	2		1.740	2.176	2	2	2		1.398	4	2		0.000	2.176	2	2	0.1	1.301	2.699
03/22/04	12:40	4.8	19.76	110	110			0.339	34	31	31		0.384										
03/29/04	12:15	9.4	34.29	2		1	1.602	2.398	2	2	2		1.845	2		1	0.000	2.398	0.1			1.301	3.146
04/05/04	13:00	6.6	11.14	11	11			1.320	2	2	2		1.097	8	8		0.138	1.459	2	2	2	0.000	1.097
04/12/04	13:00	8.9	3.85	2	2		0.602	1.176	0.1				1.301	2		1	0.000	1.176	0.1			0.000	1.301
04/19/04	13:10	13.3	27.58	8	8			2.439	0.1				4.139	2		1	0.602	3.041	0.1			0.000	4.139
04/26/04	14:52	11.8	2.87	2		1	0.000	1.176	0.1				1.903	2		1	0.000	1.176	0.1			0.000	1.903
05/10/04	11:30	13	15.66	2		1		2.398	0.1				2.924	2		1	0.000	2.398	0.1			0.000	2.924

Legend:

2	= <2 (below detection limit)
1	= dummy number to represent <2 on graph (total coliforms only)
0.1	= 0 (fecal coliforms only)
	= > (greater than) reported removal
	= < (less than) reported removal

Appendix E: Cryptosporidium Challenge Test Data

Table E.1: Water Quality Measurements during *Cryptosporidium* Challenge Test 5a

Test 5a
31-May-04
Pilot 1 (Train 1)
0.4 m/h
w/l above SSF - 104.4 cm
w/l above secondary ozone contactor - 131 cm
raw water pH ~8.32 (measured Jun 1)

Time (h)	Sampling Location	Dissolved Oxygen	Temp. (^{o}C)	Influent Turbidity (NTU)	Effluent Turbidity (NTU)	Notes
0						start of test
1	0	6.46	17.5	5.46	0.15	
1	2*	6.37	18.8	2.54		
1	3	3.98	19.5	0.16		
2	0	6.58	17.6	5.34		
2	2*	6.98	18.5	2.44		
2	3	4.2	18.8	0.15		
3	0	6.48	17.5	6.05		
3	2*	6.29	19.1	2.56		
3	3	3.75	19.7	0.15		
4	0	6.3	17.6	6.08		
4	2*	6.3	18.7	2.68		
4	3	3.73	19.2	0.15		
5	0	7.92	17.1	8.08		
5	2*	6.45	19.2	2.86		
5	3	3.53	20.3	0.15		
6	0	7.77	17.7	8.9		metering pump stopped
6	2*	6.23	18.5	3.22		
6	3	3.76	19.3	0.16		
7	0	7.75	17.9	8.52		
7	2*	6.47	18.9	3.05		
7	3	4.06	19.2	0.15		
8	0	7.39	18.3	9.24		
8	2*	6.29	19.1	2.97		
8	3	4.29	19.9	0.14		
9	0	8.21	18.3	10		
9	2*	6.47	19.1	3.15		
9	3	4.51	19.1	0.16		
	average	5.9	18.7	3.5		

Table E.2: Water Quality Measurements during *Cryptosporidium* Challenge Test 5b

Test 5b
6-Jun-04
Pilot 1 (Train 1)
0.8 m/h
w/l above SSF - 101.5
w/l above secondary ozone contactor - 151 cm
test started at 10:30 am, stopped metering pump at 4:30 pm
raw water pH ~8.43 (measured Jun 7)

Time (h)	Sampling Location	Dissolved Oxygen	Temp. (^{o}C)	Influent Turbidity (NTU)	Effluent Turbidity (NTU)	Notes
0						start of test
2	0	7.31	20.9	3.82		
2	2*	7.21	21.2	3.21		
2	3	5.35	21.9	0.19		
3	0	6.76	20.9	3.15		
3	2*	6.98	21.7	2.82		
3	3	5.21	22	0.2		
5	0	6.26	21.3	3.08		
5	2*	6.63	22.1	2.25		
5	3	4.86	22.3	0.2		
6						metering pump stopped
8	0	10.84	23	3.83		
8	2*	7.33	22.2	2.08		
8	3	5.26	22.2	0.2		
	average	6.7	21.8	2.1		

Table E.3: Water Quality Measurements during *Cryptosporidium* Challenge Test 5c

Test 5c
28-Apr-04
Pilot 2
0.4 m/h
w/l above SSF1 - 50.5 cm
w/l above SSF2 - 15.2 cm
test started at 10:10am
raw water pH 8.35

Time (h)	Sampling Location	Dissolved Oxygen	Temp. (^{o}C)	Influent Turbidity (NTU)	Effluent Turbidity (NTU)	Notes
0						start of test
1	1	11.62	10	2.09	0.26	
1	2*			0.55		
1	3	7.67	10.7	0.26		
1	4	8.45	11	0.21		
6						metering pump stopped
7	1			0.245		
7	4	8.88	12.6	0.2		
7	3	9.61	11.1	0.26		
7	2*	11.62	10.8	0.55		
7	1	15.19	9.9	1.93		
	average	10.4	10.9	0.7		

Table E.4: Actual Feedstock Concentrations Measured with a Hemacytometer

Test 5a

Hemacytometer Count	Multiplier	Concentration (oocyst/L)
9	x50,000,000	450,000,000
5	x50,000,000	250,000,000
11	x50,000,000	550,000,000
16	x50,000,000	800,000,000
5	x50,000,000	250,000,000
	average	460,000,000
	stdev	230,217,289

Test 5b

Hemacytometer Count	Multiplier	Concentration (oocyst/L)
7	x50,000,000	350,000,000
14	x50,000,000	700,000,000
10	x50,000,000	500,000,000
7	x50,000,000	350,000,000
6	x50,000,000	300,000,000
	average	440,000,000
	stdev	163,554,272

Test 5c

Hemacytometer Count	Multiplier	Concentration (oocyst/L)
8	x50,000,000	400,000,000
9	x50,000,000	450,000,000
8	x50,000,000	400,000,000
	average	416,666,667
	stdev	28,867,513

References

Aboytes, R., LeChevallier, M. (2003). Detection of Infectious *Cryptosporidium* in Filtered Drinking Water. AWWA WQTC Conference Proceedings CD-ROM, Philadelphia, Paper T6-6.

Ahmad, R., Amirtharajah, A. (1998). Detachment of Particles During Biofilter Backwashing. *Jour. AWWA*, Vol. 90(12):74-85.

Ahsan, T., Alaerts, G.J., Buiteman, J.P. (1996). Direct Horizontal-Flow Roughing Filtration. Part II: Performance, and Operational Guideline. *J Water SRT – Aqua* Vol. 45(6):281-291.

Anderson, W.B., Huck, P.M., Gaulin, T., Dorner, S.M., Slawson, R.M. (2003). Occurrence and Distribution of Selected Human Pathogens and Indicators in a Heavily Impacted Watershed. AWWA Water Quality Technology Conference Proceedings 2003, Paper T12-3.

APHA, AWWA, WEF (1998). Standard Methods for the Examination of Water and Wastewater, 20th Ed., L.S. Clesceri, A.E. Greenberg, and A.D. Eaton (Eds.), American Public Health Association, Washington, DC.

AWWA (American Water Works Association) (1981). In Proc. of the Small Water System Problems Seminar. Denver, CO, AWWA. [as cited in Collins et al. (1994)]

Baker, M.N. (1948). The Quest for Pure Water. The American Water Works Association Inc., New York.

Barrett, J.M., Silverstein, J. (1988). The Effects of High-Carbon and High-Coliform Feed Waters on the Performance of Slow Sand Filters under Tropical Conditions. In Slow Sand Filtration: Recent Developments in Water Treatment Technology, Graham, N.J.D., Ellis Horwood Ltd., England, 1998.

Barrett, J.M., Bryck, J., Collins, M.R., Janonis, B.A., Logsdon, G.S. (1991). Manual of Design for Slow Sand Filtration. AWWA Research Foundation & AWWA, Denver, CO.

Bellamy, W.D., Silverman, G.P., Hendricks, D.W., Logsdon, G.S. (1985a). Removing *Giardia* Cysts with Slow Sand Filtration. *Jour. AWWA* 77(2):52-60.

Bellamy, W.D., Silverman, G.P., and Hendricks, D.W. (1985b). Filtration of *Giardia* Cysts and Other Substances, Vol. 2. Slow sand filtration. USEPA, Cincinnati, OH, EPA-600/2-85/026.

Bellamy, W.D., Hendricks, D., and Logsdon, G.S. (1985c). Slow Sand Filtration: Influences of Selected Process Variables. *Jour. AWWA* 77(12):62.

Bellinger, E.G. (1979). *J. Instn. Wat. Engrs. Sci.*, 33:19-29. [as cited in Duncan (1988)]

Bernardo, L.D. (1988). Upflow Coarse-Grained Prefilter for Slow Sand Filtration. In Slow Sand Filtration: Recent Developments in Water Treatment Technology, Graham, N.J.D., Ellis Horwood Ltd., England, 1998.

Boller, M. (1993). Filter Mechanisms in Roughing Filters. *Journal of Water Supply Research and Technology – Aqua*, 42(3):174-185.

Brink, D.R., Parks, S. (1996). Update on Slow Sand/Advanced Biological Filtration. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

Bryck, J.M.G., Sklenar, J. (1986). Slow Sand Filtration for *Giardia* lamblia Removal: Village of 100 Mile House. Dayton & Knight Ltd., West Vancouver, B.C.

Burman, N.P. (1962). Biological Control of Slow Sand Filtration. *Effluent Water Treat. J.*, 2, 674. [as cited in Ellis (1985)]

Butkus, M.A., Bays, T.J., Labare, M.P. (2003). Influence of Surface Characteristics on the Stability of *Cryptosporidium parvum* Oocysts. *Applied and Environmental Microbiology*, July 2003, pg. 3819-3825.

Cable, C.J., Jones, R.G. (1996). Assessing the Effectiveness of Ozonation Followed by Slow Sand Filtration in Removing THM Precursor Material from an Upland Raw Water. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

Carlson, K.H., Amy, G.L. (1995). Relative Importance of EBCT and HLR on Removal of BOM during Filtration. AWWA WQTC Conference Proceedings, 1995.

Carlson, K.H., Amy, G.L., Garside, J., Blais, G. (1996). Ozone-Induced Biodegradation and Removal of NOM and Ozonation By-Products in Biological Filters. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

Clark and Regli (1991). The Basis for *Giardia* CT Values in the Surface Water Treatment Rule: Inactivation by Chlorine. USEPA, Cincinnati, Ohio. [in USEPA (1991)]

Clarke, B.A., Lloyd, B.J., Jones, C.J., Evans, H.L. (1996a). Water Treatment by Multistage Filtration Utilising Gravel Prefilters and Fabric Enhanced Slow Sand Filters. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

Clarke, B.A., Lloyd, B.J., Crompton, J.L., Major, I.P. (1996b). Cleaning of Upflow Gravel Prefilters in Multistage Filtration Water Treatment Plants. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

Cleasby, J.L., Hilmoe, D.J., Dimitracopoulos, C.J. (1984a). Slow Sand and Direct Inline Filtration of a Surface Water. *Jour. AWWA* 44-55. [as cited in USEPA (1998b)]

Cleasby, J.L., Hilmoe, D.J., Dimitracopoulos, C., and Diaz-Bossio, L.M. (1984b). Effective Filtration Methods for Small Water Supplies. EPA-600/S2-84-088, Municipal Environmental Research Laboratory, USEPA, Cincinnati, OH. [as cited in Logsdon (1991)]

Cleasby, J.L. (1991). Source Water Quality and Pretreatment Options for Slow Sand Filters. In Slow Sand Filtration. American Society of Civil Engineers, New York, Edited by G. Logsdon.

Collins, M.R., Cole, J.O., Westersund, C.M., Paris, D.B. (1994a). Assessing Roughing Filtration Design Variables. *Water Supply*, 12:1-2.

Collins, M.R., Westersund, C.M., Cole, J.O., Roccaro, J.V. (1994b). Evaluation of Roughing Filtration Design Variables. AWWA Research Foundation, Denver, CO.

Craun, G.F. et al. (1994). Balancing Chemical and Microbial Risk of Drinking Water Disinfection, Part I, Benefits and Potential Risks. *Aqua* 43(4):192-199.

Dai, X., Hozalski, R. (2002). Effect of NOM and Biofilm on the Removal of *Cryptosporidium* parvum Oocysts in Rapid Filters. *Jour. Water Research* 36(2002):3523-3532.

D'Antonio, R.G., Winn, R.E. Taylor, J.P., Gustafson, T.L., Current, W.L., Rhodes, M.M., Gary, G.W., Zajac, R.A. (1985). A Waterborne Outbreak of Cryptosporidiosis in Normal Hosts. *Ann. Int. Med.*, 103:886. [as cited in USEPA (2001)]

Datta, A.K., Chaudhuri, M. (1991) Microbial Purification in Slow Sand Filter. *Environmental Toxicology and Water Quality: An International Journal*, Vol. 6:239-247.

Droppo I.G. and Ongley, E.D. (1994). Flocculation of Suspended Sediment in Rivers of Southeastern Canada. Elsever Science Ltd., *Wat. Res.*, Vol. 28(8):1799-1809.

Droste, R.L. (1997). Theory and Practice of Water and Wastewater Treatment. John Wiley & Sons Inc., US, ISBN 0-471-12444-3.

Duncan, A. (1988). The Ecology of Slow Sand Filters. In Slow Sand Filtration: Recent Developments in Water Treatment Technology, Graham, N.J.D., Ellis Horwood Ltd., England, 1998.

Edzwald, J.K. and J.E. Van Benschoten, (1990). Aluminum Coagulation of Natural Organic Matter. Chemical Water and Wastewater Treatment. Eds: H.H. Hahn and R. Klute Springer-Verlag Berlin, pg. 341-359.

Eighmy, T.T., Collins, M.R. (1988). Modifications to the Slow Rate Filtration Process for Improved Trihalomethane Precursor Removal. In Slow Sand Filtration: Recent Developments in Water Treatment Technology, Graham, N.J.D., Ellis Horwood Ltd., England, 1998.

Eighmy, T.T., Collins, M.R., Malley, J.P. Jr., Royce, J., Morgan, D. (1993). Biologically Enhanced Slow Sand Filtration for Removal of Natural Organic Matter. AWWA Research Foundation, Denver, CO.

Ellis, K.V. (1985). Slow Sand Filtration. CRC Critical Reviews in Environmental Control, Vol. 15(4):315-354.

Ellis, K.V., Aydin, M.E. (1993). A Study of Three Slow Sand Filters at Various Flow Rates with Constant Temperature. *J Water SRT – Aqua*, Vol. 42(2):88-96.

Ellis, K.V., Aydin, M.E. (1995). Penetration of Solids and Biological Activity into Slow Sand Filters. *Wat. Res.*, Vol. 29(5):1333-1341.

Emelko, M.B. (2001). Removal of *Cryptosporidium parvum* by Granular Media Filtration. Ph.D. dissertation, University of Waterloo, Waterloo, ON.

Emelko, M.B., P.M. Huck, and I.P. Douglas (2003). *Cryptosporidium* and Microsphere Removal During Late in-Cycle Filtration. *Jour. AWWA*, 95(5):173-182.

Environment Canada (2004). Canadian Climate Normals 1971-2000. www.climate.weatheroffice.ec.gc.ca/climate_normals

Fayer, R. (1997). *Cryptosporidium* and Cryptosporidiosis. CRC Press, New York. [as cited in USEPA (2001)]

Fogel, D., Isaac-Renton, J., Guasparini, R., Moorehead, W., Ongerth, J. (1993). Removing *Giardia* and C*ryptosporidium* by Slow Sand Filtration. *Jour. AWWA*, November 1993.

Fox, K.R., Miltner, R.J., Logsdon, G.S., Dicks, D.L., and Drolet, L.F. (1984). Pilot-Plant Studies of Slow-Rate Filtration. *Journ. AWWA*, 76(12):62-68.

Fox, K.R., Graham, N.J.D. and Collins, M.R. (1994). Slow Sand Filtration Today: an Introduction Review. In Collins, M.R. and Graham, N.J.D. (eds) Slow sand filtration: an International Compilation of Recent Scientific and Operational Development. AWWA, Denver, CO, USA, p. 1-8. [as cited in Galvis et al. (1998)]

Galvis, G., Visscher, J.T., Lloyd, B. (1992). Multi-stage Surface Water Treatment for Community Water Supply in Columbia. *Waterlines*, Vol. 10(3):26-29.

Galvis, G., Fernandez, J., Visscher, J.T. (1993). Comparative Study of Different Pre-Treatment Alternatives. *J. Water SRT-Aqua*, Vol. 42(6):337-346.

Galvis, G., Latorre, J., Ochoa, A.E., Visscher, J.T. (1996). Comparison of Horizontal and Upflow Roughing Filtration. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

Galvis, G., Latorre, J., Visscher, J.T. (1998). Multi-stage Filtration: An Innovative Water Treatment Technology. IRC International Water and Sanitation Centre, The Hague, Netherlands, TP Series, No. 34E.

Galvis, G., Visscher, J.T. (1999). Multistage Filtration: A Practical, Low-Cost Technology. In Providing Safe Drinking Water in Small Systems, Contruvo, J.A., Craun, G.F., Hearne, N., CRC Press LLC, 1999.

Galvis, G., Latorre, J., Galvis, A. (2002). Multi-stage Filtration Technology. In Small Community Water Supplies. IRC Technical Paper Series 40. IRC International Water and Sanitation Centre, 2002.

Giotom, K. (1990). Performance of Pilot Scale Slow Sand Filters using Different Local Sands in Ethiopia. Institute of Water and Environmental Engineering, Tampere University of Technology.

GRCA (2004). Conserving Our Future (brochure). Grand River Conservation Authority. http://www.grandriver.ca/grca/pdf/GRCABrochure.pdf

Greaves, G.F., Grundy, P.G., Taylor, G.S. (1988). Ozonation and Slow Sand Filtration for the Treatment of Coloured Upland Waters – Pilot Plant Investigations. In Slow Sand Filtration: Recent Developments in Water Treatment Technology, Graham, N.J.D., Ellis Horwood Ltd., England, 1998.

Gould, M.H., Cameron, D.A., Zabel, T.F. (1984). An Experimental Study of Ozonation Followed by Slow Sand Filtration for the Removal of Humic Colour from Water. *Ozone Sci. Eng.*, 6:3-15. [as cited in Eighmy et al. (1993)]

Haarhoff, J. and Cleasby, J.L. (1991). Biological and Physical Mechanisms in Slow Sand Filtration. In Slow sand filtration. American Society of Civil Engineers. Edited by G. Logsdon, Reston, Va.

Health Canada (2003). Summary of Guidelines for Canadian Drinking Water Quality. Federal-Provincial-Territorial Committee on Drinking Water.

Hendel, B., Marxsen, J., Fiebig, D., Preub, G. (2001). Extracellular Enzyme Activities during Slow Sand Filtration in a Water Recharge Plant. *Wat. Res.*, Vol. 35(10):2484-2488.

Huck, P.M. (1988). Reduction in Organic Levels and Disinfection Demand by Slow Sand Filtration in Western Europe. Alberta, Canada, Department of Civil Engineering, University of Alberta. [as cited in Galvis et al. (1998)]

Huck, P.M., Finch, G.R., Hrudey, S.E., Peppler, M.S. (1998). Design of Biological Processes for Organics Control. AWWA Research Foundation, Denver, CO.

Huck, P.M., Coffey, B.M., Amirtharajah, A., Bouwer, E.J. (2000). Optimizing Filtration in Biological Filters. AWWA Research Foundation, Denver, CO.

Huck, P.M., B.M. Coffey, C.R. O'Melia and M.B. Emelko and D.D. Maurizio (2001). Filtration Operation Effects on Pathogen Passage. American Water Works Association Research Foundation and American Water Works Association, Report No. 90874. Denver, Colorado, ISBN 1-58321-170-5.

Huck, P.M., B.M. Coffey, M.B. Emelko, D.D. Maurizio, R.M. Slawson, W.B. Anderson, J. Van Den Oever, I.P. Douglas and C.R. O'Melia (2002). Effects of Filter Operation on *Cryptosporidium* Removal. *Jour. AWWA*, 94(6):97-111.

Hudson, H.E. Jr. (1981). Water Clarification Processes: Practical Design and Evaluation, Van Nostrand Reinhold Company, New York.

Huisman, L., Wood, W.E. (1974). Slow Sand Filtration. World Health Organization, Geneva, Switzerland.

Huisman, L. (1977). Slow Sand Filtration. Delft University of Technology, The Netherlands, p. 5-6. [as cited in Goitom (1990)]

Hrubec et al. (1991). Behaviour of Some Substituted Benzenes, Pesticides and Synthetic Agents during SSF. In H_20, Vol. 24(13):348-351. [as cited in Galvis et al. (1998)]

Hsu, B.M., Huang, C., Pan, J.R. (2001). Filtration Behaviours of *Giardia* and *Cryptosporidium* – Ionic Strength and pH Effects. Elsevier Science Ltd., Great Britain, *Wat. Res.*, Vol. 35(16):3777-3782.

Ingallinella, A.M., Stecca, L.M., Wegelin, M. (1998). Up-flow Roughing Filtration: Rehabilitation of a Water Treatment Plant in Tarata, Bolivia. Elsevier Science Ltd, Great Britain, *Wat. Sci. Tech.*, Vol. 37(9):105-112.

IRC (International Water and Sanitation Centre) (1989). Pretreatment Methods for Community Water Supply. J. Smet and J.T. Visscher, Eds. The Hague, Netherlands: IRC. [as cited in Collins et al. (1994b)]

Iwasaki, T. (1937). Some Notes on Sand Filtration. *Jour. AWWA*, 29, 1591 [as cited in Clarke et al. (1996a)]

Jekel, M.R. (1986). Interaction of Humic Acids and Aluminium Salts in the Flocculation Process. *Water Res.*, 1986, No. 20(12):1535-1543. [as cited in Ahsan et al. (1996)]

Karanfil, T., Schlautman, M.A., Erdogan, I. (2002). Survey of DOC and UV measurement Practices with Implications for SUVA Determination. *Jour. AWWA*, 94(12).

Kimber, M. (2003). Cleaner Waters. Royal Society of Chemistry (RSC) http://www.chemsoc.org/chembytes/ezine/2003/kimber_may03.htm

Korshin, G.V., Li, C.W., Benjamin, M.M. (1997). Monitoring the Properties of Natural Organic Matter Through UV Spectroscopy: A Consistent Theory. *Water Resources*, 31(7):1787. [as cited in Karanfil et al., 2002]

Lambert, S.D., Graham, J.D. (1995). A Comparative Evaluation of the Effectiveness of Potable Water Filtration Processes. *J. Water SRT – Aqua*, vol. 44(1):38-51.

LeChevallier, M.W., Norton, W.D. (1992). Examining Relationships Between Particle Counts and *Giardia*, *Cryptosporidium*, and Turbidity. *Jour. AWWA*, 84:12:54-50.

LeChevallier, M.W.; Norton, W.D. (1995). *Giardia* and *Cryptosporidium* in Raw and Finished Water. *Jour. AWWA*, 87:9:54.

LeChevallier, M.W., Di Giovanni, G., Clancy, J.L., Bukhari, Z., Bukhari, S., Hargy, T., Rosen, J.S., Sobrinho, J., Frey, M.M. (2000). Source Water Assessment: Variability of Pathogen Concentrations. AWWA Water Quality Technology Conference Proceedings 2000.

LeChevallier, M.W., Di Giovanni, G., Clancy, J.L., Bukhari, Z., Bukhari, S., Hargy, T., Rosen, J.S., Sobrinho, J., Frey, M.M. (2002). Source Water Assessment: Variability of Pathogen Concentrations. AWWARF, Project #488, Denver, CO.

LeCraw, R.A., Abernathy, R., Collins, M.R., Huck, P.M., Anderson, W., Cleary, S. (2004). Pre-ozonation and Roughing Filter Enhancements to Slow Sand Filtration. 11th National Conference and 2nd Policy Forum on Drinking Water, Calgary, April 2004.

LeCraw, R.A. (2004). personal communication. MS Filter Inc., rlecraw@msfilter.com

Lippy, O.H. and Waltrip, S.C. (1984). Waterborne Disease Outbreaks-1946-1980: A Thirty-Five Year Perspective. *Jour. AWWA*, 76(2):60. [as cited in Collins et al., 1994]

Lloyd, B. (1973). The Construction of a Sand Profile Sampler: Its Use in the Study of the Vorticella Populations and General Intestinal Microfauna of Slow Sand Filter. *Water Res.*, 7:963-973. [as cited in Datta and Chaudhuri (1991)]

Lloyd, B. (1974). The Functional Microbial Ecology of Slow Sand Filters. PhD thesis, University of Surrey, UK. [as cited in Galvis et al. (1992)]

Lloyd, B.J., Wheeler, D.C., Baker, T. (1983). The Evaluation and Development of a Treatment System for Surface Water. *The Public Health Engineer*, October 1983, Vol. 11(4):17-22.

Lloyd, B.J. (1996). The Significance of Protozoal Predation and Adsorption for the Removal of Bacteria by Slow Sand Filtration. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

Logan, A.J., Stevik, T.K., Siegrist, R.L., Ronn, R.M. (2001). Transport and Fate of *Cryptosporidium* parvum Oocysts in Intermittent Sand Filters. Elsevier Science Ltd., Great Britain, *Wat. Res.*, Vol. 35(18):4359-4369.

Logsdon, G.S., and Lippy, E.C. (1982). The Role of Filtration in Preventing Waterborne Disease. *Jour. AWWA*, 74(12):649-655. [as cited in Schuler and Ghosh (1991)]

Logsdon, G.S. (1991). Slow Sand Filtration: A Report Prepared by the Task Committee on Slow Sand Filtration. American Society of Civil Engineers, New York.

Logsdon, G.S. et al. (1993). Control of *Giardia* Cysts by Filtration: The Laboratory's Role. Proc. 1983 AWWA WQTC, Norfolk, Va. [as cited in Fogel et al. (1993)]

Mallevaille, J., Duguet, J.P. (1988). Comparisons Between Activated Carbon and Slow Sand Filtration in the Treatment of Surface Waters. In Slow Sand Filtration: Recent Developments in Water Treatment Technology, Graham, N.J.D., Ellis Horwood Ltd., England, 1998.

Mbwette, T.S.A., Wegelin, M. (1989). Design, Operation and Maintenance Considerations of Horizontal Flow Roughing Filters. *Waterlines*, Vol. 8(1):27-29.

McConnell, L.J. (1984). Evaluation of the Slow Rate Sand Filtration Processes for Treatment of Drinking Water Containing Viruses and Bacteria. M.Sc. thesis. Logan, UT, USA, Utah State University. [as cited in Galvis et al. (1998)]

McMeen, C.R., Benjamin, M.M. (1996). Removal of Natural Organic Matter by Slow Sand Filtration Through Iron-Oxide-Coated-Olivine. In Slow Sand Filtration: Recent Developments in Water Treatment Technology, Graham, N.J.D., Ellis Horwood Ltd., England, 1998.

Melin, E.S., Odegaard, H. (2000). The Effect of Biofilters Loading Rate on the Removal of Organic Ozonation By-Products. *Wat. Res.*, Vol. 34(18):4464-4476.

MOE (2003). Procedure for Disinfection of Drinking Water in Ontario (as adopted by reference by Ontario Regulation 170/03 under the Safe Drinking Water Act). Ontario Ministry of Environment.

Mogren, E.M., Scarpino, P., Summers, R.S. (1990). Measurement of Biodegradable Dissolved Organic Carbon in Drinking Water. In Proc. of the 1990 AWWA Annual Conference. AWWA, Denver, CO. [as cited in Eighmy et al. (1993)]

Moll, D.M., Summers, R.S. (1996). Performance, Biomass and Community Structure Profiles of Biological Rapid Media Filters. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996

Montgomery, J.M. (1985). Water Treatment: Principles and Design. John Wiley & Sons.

Moran, M.; Moran, D.; Cushing, R.S.; Lawler, D.F. (1993). Particle Behaviour in Deep Bed Filtration: Part 2-Particle Detachment. *Jour. AWWA*, 85:12:82-93.

Muhammed, N., Ellis, K., Parr, J., Smith, M.D. (1996). Optimization of Slow Sand Filtration. Reaching the Unreached: Challenges for the 21st Century, WEDC Conference, WEDC, Loughborough University, Leicestershire, pg. 283-285.

Müller, M.B., Fritz, W., Lankes, U., Frimmel, F.H. (2004). Ultrafiltration of Nonionic Surfactants and Dissolved Organic Matter. *Environ. Sci. Technol.*, 2004(38):1124-1132.

Nieminski, E.C., Ongerth, J.E. (1995). Removing *Giardia* and *Cryptosporidium* by Conventional Treatment and Direct Filtration. *Journ. AWWA*, 87(9):96-106. [as cited in USEPA (2001)]

Ongley, E.D. (1974). Hydrophysical Characteristics of Great Lakes Tributary Drainage, Canada. Pollution and Land Use Activities Reference Group (PLUARG), Task D, Activity 1, Sub-Activity 1. International Joint Commission, Windsor, Ontario, Canada. [as cited in Droppo and Ongley (1994)]

Owen, D.M., Amy, G.L., Chowdury, Z.K. (1993). Characterization of Natural Organic Matter and its Relationship to Treatability. AWWA Research Foundation, Denver, CO. [as cited in Karanfil et al., 2002]

Peldszus, S., Huck, P.M., Andrews, S.A. (1996). Determination of Short-Chain Aliphatic, Oxo- and Hydroxy-Acids in Drinking Water at Low Microgram per Liter Concentrations. *J. Chromatography A*, Vol. 723:27-34.

Pontius, F.W. (1995). *Cryptosporidium*: Answers to Common Questions. *Jour. AWWA*, 87(9):10.

Poynter, S.F.B. and Slade J.S. (1977). The Removal of Viruses by Slow Sand Filtration. *Progress in Water Technology, A Journal of the International Association of Water Pollution Research*, 9(1):75-88.

Rachwal, A.J., Bauer, M.J., West, J.T. (1998). Advanced Techniques for Upgrading Large Scale Slow Sand Filters. In Slow Sand Filtration: Recent Developments in Water Treatment Technology, Graham, N.J.D., Ellis Horwood Ltd., England, 1998.

Rajapakse, J.P., Ives, K.J. (1990). Filtration of Very Highly Turbid Waters using Pebble Matrix Filtration. *Jour. IWEM*, 1990, 4, April.

Rittmann, B.E., and Huck, P.M. (1989). Biological Treatment of Public Water Supplies. *CRC Crit. Rev. Environ. Control*, 19(2):119-184. [as cited in Eighmy et al., 1993]

Rollinger, Y., and Dott, W. (1987). Survival of Selected Bacterial Species in Sterilized Activated Carbon Filters and Biological Activated Carbon Filters. *Appl. Environ. Microbiol*., 23:234-243. [as cited in Eighmy et al. (1993)]

Saidam, M.Y., Butler, D. (1996). Algae Removal by Horizontal Flow Rock Filters. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

Sattar S.A.; Chauret, C.; Springthorpe, V.S. (1999). *Giardia* cyst and *Cryptosporidium* Oocyst Survival in Watersheds and Factors Affecting Inactivation. AWWARF & AWWA, ISBN 0-89867-975-3, pg. 75-107.

Schuler, P.F., Ghosh, M.M., and Boutros, S.N. (1988). Comparing the Removal of *Giardia* and *Cryptosporidium* using Slow Sand and Diatomaceous Earth Filtration. Proc. 1988 AWWA Ann. Conf., Orlando, Fla. [as cited in Fogel et al. (1993)]

Schuler, P.F., Ghosh, M.M. (1991). Slow Sand Filtration of Cysts and Other Particulates. AWWA Annual Conference Proceedings 1991, AWWA, Denver, CO, pg. 235-252.

Schulz, C.R., Okun, D.A. (1984). Surface Water Treatment for Communities in Developing Countries. John Wiley & Sons, New York.

Seelaus, T.J., Hendricks, D.W., and Janonis, B.A. (1986). Design and Operation of a Slow Sand Filter. *Jour. AWWA*, 78(12):35-41. [as cited in Schuler and Ghosh (1991)]

Seger, A. and Rothman, M. (1996). Slow Sand Filtration With and Without Ozonation in Nordic Climate. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

Sims, R.C., Slezak, L.A. (1991). Slow Sand Filtration: Present Practice in the United States. In Slow Sand Filtration. American Society of Civil Engineers, New York, Edited by G. Logsdon.

Stephenson, P., Benedek, A., Malaiyandi, M., and Lancaster, E.A. (1980). The Effect of Ozone on the Biological Degradation and Activated Carbon Adsorption of Natural and Synthetic Organics in Water. Part 1. Ozonation and Biodegradation. International Ozone Association, *Ozone: Science and Engineering*, Vol. 1, pp. 263-279, 1979.

Thanh, N.C., Hettiaratchi, J.P.A (1982). Surface Water Filtration for Rural Areas: Guidelines for Design, Construction, and Maintenance. Environmental Sanitation Information Centre, Bangkok, Thailand.

Timms, S., Slade, J.S., Fricker, C.R. (1995). Removal of *Cryptosporidium* by Slow Sand Filtration. *Wat. Sci. Tech.*, 31:5-6:81-84.

Tipping, E., Higgins, D.C. (1982). The Effect of Adsorbed Humic Substances on the Colloid Stability of Haematite Particles. *Colloid Surf*, 1982, No. 5:85-92. [as cited in Ahsan et al. (1996)]

Troyan, J.J, Hansen, S.P. (1989). Treatment of Microbial Contaminants in Potable Water Supplies. Noyes Data Corporation, Park Ridge, NJ. [as cited in Yahya et al. (1993)]

Uhl, W., Gimbel, R. (1996). Investigations on the Performance of Fast-Rate Biological Filters in Drinking Water Treatment. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

USEPA (1991). Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Systems using Surface Water Sources. Prepared by Malcolm Pirnie Inc. and HDR Engineering Inc., USEPA, Washington, D.C.

USEPA (1996). ICR Microbiology Laboratory Manual. Office of Research and Development, Washington, D.C.

USEPA (1997). Small System Compliance List for the Surface Water Treatment Rule. EPA 815-R-97-002.

USEPA (1998a). National Primary Drinking Water Regulations: Disinfectants and Disinfection Byproducts: Final Rule. RIN 2040-AB82, 40 CFR Parts 9, 141 and 142, Section IIE, December 1998.

USEPA (1998b). *Giardia*: human health criteria document. USEPA, Washington, D.C., EPA-823-R-002.

USEPA (1999). *Giardia*: drinking water health advisory. USEPA, Washington, D.C., EPA-822-R-99-008.

USEPA (2001a). *Cryptosporidium*: drinking water health advisory. USEPA, Washington, D.C., EPA-822-R-01-009.

USEPA (2001b). *Cryptosporidium*: human health criteria document. USEPA, Washington, D.C., EPA-822-K-94-001

Van der Hoek, J.P., Bonné, P.A.C., Kors, L.J., Te Welscher, R.A.G. (1996). Slow Sand Filtration: Effect of Grain Size and Filtration Rate on Operation and Performance. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

Van Dijk, J.C., Ooman, J.H.C.M. (1978). Slow Sand Filtration for Community Water Supply in Developing Countries: A Design and Construction Manual. WHO International Reference Centre for Community Water Supply, The Hague, Netherlands, Chap. 4. [as cited in Ellis (1985)]

Visscher, J.T., Paramasivam, R., Raman, A., Heijnen, H.A. (1987). Slow Sand Filtration for Community Water Supply: Planning, Design, Construction, Operation and Maintenance. Technical paper series; No. 24. The Hague, The Netherlands, IRC International Water and Sanitation Centre.

Weber-Shirk, M.L., Dick, R.I. (1997a). Biological Mechanisms in Slow Sand Filters. *Jour. AWWA*, Vol. 89(2):72-83.

Weber-Shirk, M.L., Dick, R.I. (1997b). Physical-Chemical Mechanisms in Slow Sand Filters. *Jour. AWWA*, Vol. 89(1):87-100.

Weber-Shirk, M.L., Dick, R.I. (1999). Bacterivory by a Chrysophyte in Slow Sand Filters. *Wat. Res.*, Vol. 33(3):631-638.

Weber-Shirk, M.L. (2002). Enhancing Slow Sand Filter Performance with an Acid-Soluble Seston Extract. *Water Research*, Vol. 36(2002):4753-4756.

Wegelin, M. (1983). Roughing Filters as Pre-Treatment for Slow Sand Filtration. *Wat. Supply*, Vol. 1:67-75.

Wegelin, M., Boller, M. and Schertenleib, R. (1986). Particle Removal by Horizontal-Flow Roughing Filtration. *Aqua*, Vol. 35(3):115-125.

Wegelin, M. (1988). Roughing Gravel Filters for Suspended Solids Removal. In Slow Sand Filtration: Recent Developments in Water Treatment Technology, Graham, N.J.D., Ellis Horwood Ltd., England, 1998.

Wegelin, M., Schertenleib, R. (1993). Roughing Filters for Water Treatment. International Reference Centre for Waste Disposal, IRCWD News, No. 27, August 2003.

Wegelin, M. (1996). Surface Water Treatment by Roughing Filters: A Design, Construction, and Operation Manual. Sandec Report No. 2/96. Swiss Centre for Development Cooperation in Technology and Management (SKAT), St. Gallen, Switzerland.

Welté, B., Montiel, A. (1996). Removal of BDOC by Slow Sand Filtration: Comparison with Granular Activated Carbon and Effect of Temperature. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

White, M.C. et al. (1997). Evaluating Criteria for Enhanced Coagulation Compliance. *Jour. AWWA*, 89(5):64. [as cited in Karanfil et al., 2002]

WHO and UNICEF (2000). Global Water Supply and Sanitation Assessment 2000 Report. World Health Organization and United Nations Children's Fund.

Williams, P.G. (1987). A Study of Bacteria Reduction by Slow Sand Filtration. Paper presented at the 1987 IWPC Biennial Conference, Port Elizabeth. National Institute for Water Research, Pretoria, South Africa.

Wolters, H., Smet, J., Pardon, G. (1989). Downflow Roughing Filtration. In Pretreatment Methods for Community Water Supply. J. Smet and J.T. Visscher, eds. The Hague, Netherlands: IRC. [as cited in Collins et al. (1994b)]

Woolschlager, J. and Rittman, B.E. (1995). Evaluating What is Measured by BDOC and AOC Tests. *Rev. Sci. Eau*, 8:371-385.

Wricke, B., Petzoldt, H., Heiser, H., Bornmann, K. (1996). NOM – Removal by Biofiltration After Ozonation – Results of a Pilot Plant Test. In Advances in Slow Sand and Alternative Biological Filtration. Graham, N., Collins, R., John Wiley & Sons Ltd., England, 1996.

Yahya, M.T., Cluff, C.B., Gerba, C.P. (1993). Virus Removal by Slow Sand Filtration and Nanofiltration. IAWQ, Great Britain, *Wat. Sci. Tech.*, Vol. 27(3-4):445-448.

Zabel, T.F. (1985). The Application of Ozone for Water Treatment in the United Kingdom – Current Practice and Recent Research. *Ozone Sci. Eng.*, 7:11-46. [as cited in Eighmy et al. (1993)]