

**Modification of a Biosand Water Filter for
Household Treatment of High Turbidity Water**

by

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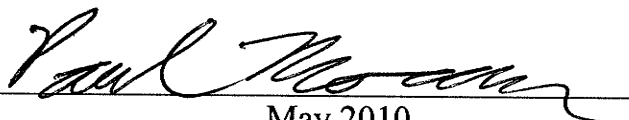
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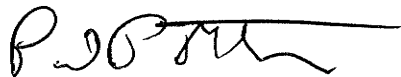
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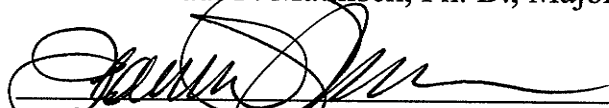
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Abstract

One billion of the poorest people in the world today do not have access to improved drinking water. Without treatment, fecal contamination results in an overwhelming disease burden. A long term best practice solution will take decades to implement. In the meantime, approximately 5 million children under five die each year from gastrointestinal diseases. This tragedy can be alleviated by household water treatment.

Household Water Treatment and safe Storage systems (HWTS) provide an interim solution. While many low cost and simple technologies exist, none of them are effective against high suspended solids concentrations (>50 NTU). Previous short-term field research by others has considered modifying a BioSand water Filter (BSF), to include pretreatment through an upper sand layer in order to extend the run cycle of the primary filter, enabling complete ripening to occur.

In this research program, one control and twelve configurations of modified filters were setup in the laboratory. Water was chemically conditioned to provide worst case scenario treatment by adjusting pH, TDS, and particle dispersion. Sample water was passed through each filter daily, and monitored for DO, turbidity, flow rate, and *E. coli* concentrations.

The results indicate that pretreatment is not necessarily beneficial under all water quality conditions. Recommendations include a description of conditions under which the modification may be beneficial, and optimized pretreatment design criteria. Regardless of water quality conditions, it was found that changing the operational guidelines for filter use can significantly improve treatment efficiency, without complicating the filter design. Design guidelines for an unmodified filter coupled with operational guidelines are provided, in order to obtain sufficient quantities of the best possible water quality under high turbidity conditions. This will enable the BSF to be used in high turbidity conditions and still significantly improve the drinking water quality. It is hoped that this will decrease the disease burden and loss of life in many of the world's poorest communities.

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List of Abbreviations

BSF	Biosand Water Filter
DALY	Disability Adjusted Life-Year
DO	Dissolved Oxygen
DSL	Dual Sand Layer
HWTS	Household Water Treatment and safe Storage
NTU	Nephelometric Turbidity Units
PV	Pore Volume
SSF	Slow Sand Filtration
SSL	Single Sand Layer
WHO	World Health Organization

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1 Introduction

Access to safe drinking water is a fundamental human need. Approximately 1 billion people in the world today do not have access to safe drinking water (United Nations, 2008). The resulting disease burden and loss of life is one of the most urgent problems facing humanity. In fact, it is estimated that 5 million children under five die each year due to gastrointestinal diseases; a significant fraction of this is directly due to the consumption of contaminated untreated water (OECD, 2003). Many technologies are available which can effectively treat any type of source water at regional or municipal scales. However, development of safe drinking water infrastructure requires years of work, skilled labor, significant investment and skilled operation. A rapid, simple, and cost-effective solution is needed.

Water treatment at the household scale is the most promising immediate solution. This provides safe water at the point of use, eliminating many opportunities for recontamination. Several robust and inexpensive technologies have already been developed, such as chlorine disinfection tablets, solar disinfection, ceramic pot filters, and Biosand filters (Stevenson, 2008). All of these can either be manufactured locally, or distributed at low cost. They provide varying degrees of treatment, generally focused on particle removal and deactivation of pathogens. For example, the Biosand Filter (BSF) uses slow sand filtration technology, modified for intermittent use in order to enable treatment at the household scale (Manz, 2007). The critical modification consists of a hydraulic control on the water flow such that 5 cm of water is maintained above the sand during periods of disuse. This supernatant water layer permits oxygen transfer to the biologically active sand layer (Manz, 2007). High turbidity water disrupts treatment by causing excessive clogging, which necessitates frequent maintenance. This in turn disrupts both the biological treatment and the removal of particles.

Many areas of the world suffer from excessively turbid source waters. Frequently, the best water available has turbidity of 100 NTU or more, with measured values exceeding 2000 NTU (Losleben, 2008). However, the widely used household treatment technologies are not recommended for use with source water with turbidity less than approximately 50 NTU. More turbid water generally hinders disinfection, causes harmful byproducts, or increases maintenance

frequency to interfere with treatment. The currently available treatment technologies for high turbidity waters are expensive and technically challenging, making household treatment difficult.

The purpose of this project is to provide an improved low cost option for the household treatment of high turbidity waters. This research examined the feasibility of treating high turbidity source waters using a modified Biosand Filter (BSF). The specific modification examined is the addition of a dual sand layer as a simple pretreatment technology, in order to protect the BSF itself from excessive clogging and maintenance. This modification was proposed and initially examined by others, primarily in the field (Collin, 2009; Kikkawa, 2008). This project supplements their work with more extensive laboratory testing of a variety of design parameters.

The approach included the development of five laboratory filters, which were designed and constructed at the bench scale; one as a control and the other four as modifications with varied design parameters. Daily operation of the filters was conducted in three phases, with different configurations of the modified filters for each phase. Turbidity, *E. coli*, dissolved oxygen, and flow rate were measured regularly to gauge the performance of each filter, with additional tests conducted periodically. The results include an assessment of filter performance, with a characterization of the impacts of filter operational parameters on water quality. These results are used to develop a set of guidelines for ensuring sufficient water quantity and quality are maintained under high turbidity conditions.

The report begins with an overview of the background information in Chapter 2, including the need for safe water in many areas of the world, the interim solution of household water treatment, an overview of existing water treatment systems, and an introduction into the BSF as a modification of slow sand filtration for intermittent use. These topics are presented as a basis for examining a modification to the BSF. Next, Chapter 3 presents the laboratory methodology utilized, including the parameters examined and the test methods. A detailed description is provided of the challenges associated with duplicating stable high turbidity source waters and the successful method employed to overcome that challenge.

The results of the filter experiments are presented in Chapter 4, with targeted observations highlighted. The effects of pH and ionic concentration on particle stability and their importance in the manufacture of stable high turbidity water are discussed. The change in filter performance due to each dual sand layer design parameter is analyzed, including the presence of the layer itself. More significantly, the impacts of filter operational guidelines for both modified and unmodified filters on water quality are observed and discussed.

Finally, Chapter 5 contains specific useful recommendations that are based on the laboratory observations. A simple test to determine whether the dual sand modification may outperform an unmodified BSF is described. A series of design parameters for the dual sand layer are proposed. Regardless of whether the dual sand layer is chosen, a set of guidelines for obtaining the best possible water quality out of any BSF are suggested, along with design parameters that optimize the effective use guidelines to ensure sufficient water quantity and quality.

2 Background

2.1 The Need for Clean Water

In the world today, nearly one billion people lack access to an improved water source (United Nations, 2008). The situation is greatly improved from 20 years ago; since then, 1.6 billion people have gained access to an improved source (United Nations, 2008). Improved water sources are defined as household piped connections, public standpipes, boreholes, protected dug wells, protected springs, and rainwater collection systems (WHO, 2008). All other sources are considered unimproved, including unprotected wells, unprotected springs, vendor-provided water, bottled water, tanker truck provision of water, and untreated streams and lakes. Many who have access to improved water suffer from intermittent supply. Recontamination of water is common, especially when water is not supplied at the point of use.

As shown in the next section, even an improved water supply does not necessarily always supply “safe” water. Safe water “does not represent any significant risk to health over a lifetime of consumption, including different sensitivities that may occur between life stages” (WHO, 2008). While much has been accomplished in providing safe water to the entire world, there is still much work to be done.

2.1.1 Gastrointestinal Diseases

One of the primary dangers regarding the ingestion of drinking water is the risk of infection due to pathogens (OECD, 2003). These pathogens consist of bacteria, viruses, protozoa, and helminths. There is a wide variety of infectious diseases that result from waterborne pathogens. The most common are enteric infections, which are often referred to as gastrointestinal or diarrheal diseases. In addition, certain waterborne pathogens lead to hepatitis E, hemorrhagic colitis, and hemolytic uremic syndrome, among others (OECD, 2003).

Waterborne disease occurrences can be classified into two categories: outbreaks and sporadic. Historically, outbreaks have been of particular concern, because a single contaminated water source can infect the population of an entire city. The most often cited example is the outbreak of

cryptosporidiosis in Milwaukee, Wisconsin in 1993 (OECD, 2003). It was estimated that 400,000 individuals suffered symptoms.

Sporadic infections are much more difficult to detect than outbreaks, but are perhaps no less important. Most cases of gastrointestinal illness have relatively mild symptoms, and may go unreported. Those that are reported can usually be spread through several routes, and will likely not be traced to the water supply (OECD, 2003). Therefore, to measure the disease burden and the impact of treating the water, estimates and case studies are the best available data.

Enteric infections are known to be one of the world's most common causes of death, especially in the developing world. In poor countries, one study listed diarrheal diseases third, beneath lower respiratory infections and HIV/AIDS, but above malaria and tuberculosis. The same study recorded 250 times more deaths due to diarrheal diseases in poor countries than in rich countries (Glennister *et al.*, 2007).

Children under the age of five are most susceptible to gastrointestinal disease. The actual worldwide statistics are difficult to measure, but it is estimated that children under five suffer from 1.4 billion cases of diarrhea annually. Approximately 4.9 million of those cases are fatal. While this is a small fatality rate per infection (0.35%), the prevalence is overwhelming (OECD, 2003).

Most gastrointestinal pathogens are transmitted through several pathways, including water, food, and person-to-person interaction. For people who do not have access to an improved water source, approximately one third to one half of all gastrointestinal infections can be attributed to waterborne pathogens. Similar results have been obtained from direct estimates, as well as from observations of the disease burden in a population before and after improvement of the drinking water source (OECD, 2003; Stauber *et al.*, 2007). The percentage of diseases due to waterborne pathogens can fluctuate significantly based on the quality of the water and the sanitary and hygienic practices of the individual.

In summary, safe water is an incredibly important need in the world today. If the entire developing world were supplied with microbiologically safe drinking water, the disease burden would be dramatically reduced. The direct results would be decreased infant mortality, and increase life expectancy. In addition, school children would spend less time staying sick at home and more time learning and working adults would be more productive.

2.1.2 Case Country: Ghana Water Quality

Much of the previous work upon which this research is founded has taken place in Northern Region, Ghana. In particular, the feed water quality and the Biosand Filter design and operation were considered based on the conditions and current practices reported for Northern Region, Ghana. Accordingly, the specific results of this research program are expected to particularly apply to this area of the world. Nevertheless, the recommendations are expected to generally apply to any poor community with highly turbid drinking water.

The theses of Massachusetts Institute of Technology students Clair Collin (2009), Matt Stevenson (2008), and Izumi Kikkawa (2008) provide useful summaries and first-hand descriptions of the conditions prevalent in Northern Region, Ghana. Based on the information they provide, unsafe drinking water is prevalent in this region. Some localities have piped drinking water; in many cases this piping system is outside of the home, enabling recontamination between the pipe and the point of use. In addition, piped water supplies frequently suffer from intermittent service, rarely providing water 24 hours a day, 7 days a week. Many other people depend on boreholes, wells, and spring water, which often provide high quality water. However, in many districts up to 75% of the population only have access to streams and dams known as “dugouts”, which are often more than a 30 minute walk away from these poor rural households. Streams and dugouts are frequently both highly contaminated and highly turbid. Turbidity in these sole-source water supplies has been measured to exceed 2,000 NTU, with typical values ranging from 100 to 1,000 NTU.

2.1.3 Acceptable Risk and Drinking Water Standards

The definition of safe water quoted in Section 2.1 states that there should be no “significant risk” associated with drinking safe water. In fact, there is always some measureable risk from drinking

water. The World Health Organization (WHO) Guidelines for Drinking-water Quality suggest that a risk of 10^{-6} DALY should be a typical standard for drinking water contamination (2008). The Disability Adjusted Life-Year (DALY) is a unit of measure which accounts for the equivalent number of years lost by each person who is afflicted in any way, adjusted to account for the severity of each symptom. A risk of 10^{-6} DALY means that among a population of one million people, a total of one year of life would be lost. It is mathematically the same whether that entire year is suffered by one person or spread out over a much larger population; in either case, the total effect on the population is minimal.

Although the WHO suggests 10^{-6} DALY as an acceptable risk, it is acknowledged that in many cases this standard of treatment is excessively high. Higher drinking water standards mean that more money must be spent to treat the same amount of water. For example, the overall disease burden of a nation may be so high that treating the water beyond a standard of 10^{-4} DALY results in health benefits that are negligibly small compared to other, larger sources of disease. As described in Section 2.1.1, waterborne diseases account for about one-third of gastrointestinal infections. Once the vast majority of waterborne pathogens and contaminants are removed, consideration of the concept of acceptable risk would suggest that it is better to spend more money on other programs, such as hygienic education and sanitary systems, rather than continuing to invest in a treatment system which is good enough for the moment.

2.2 Water Quality Parameters

There are many important aspects of water quality depending on the use planned for the water sample. In this research program, three water quality parameters received primary focus: *E. coli*, turbidity, and Dissolved Oxygen (DO). *E. coli* concentration was used as an index of microbiological treatment efficiency. Turbidity was used as a surrogate measurement of suspended solids. DO is particularly important to the microbiological treatment mechanism utilized by the Biosand Filter, and was used to ensure that anaerobic conditions do not occur as a result of modifications.

2.2.1 Escherichia coli

This section describes the science behind measurement of the microbial water quality. This provides justification for why *E. coli* has been selected as an index organism, what conclusions can be made based on *E. coli* measurements, and the precautions for interpreting and applying the data.

The microbial safety of drinking water depends on the absence of pathogens. Waterborne pathogens consist of bacteria, viruses, protozoa, and helminths. The World Health Organization has included in their Guidelines for Drinking-water Quality a set of microbial fact sheets (WHO, 2008). These fact sheets describe seventeen categories of bacteria pathogens, seven categories of viral pathogens, ten of protozoan pathogens, and three of helminth pathogens.

In order to determine the safety of drinking water, a water supply could be tested for each and every known pathogen. However, this process would be quite expensive and time consuming due to the sheer number of existing pathogens and the complexity of the necessary testing methods. When all the tests were finally complete, the water quality might be entirely different from when the testing began. In addition, there are most likely many unknown pathogens, which would elude such a procedure. For these reasons, simpler, less exhaustive tests have been used historically and are currently built into the regulatory framework for drinking water standards around the world (OECD, 2003).

The common source of nearly all waterborne microbial pathogens is human and animal feces. Waterborne pathogens can survive in the environment, but they generally do not multiply until they have infected a suitable host, such as the intestines of a warm-blooded animal. Infected people typically excrete pathogens into wastewater. This wastewater might then be subject to varying degrees of treatment, and is eventually reintroduced into either surface water or groundwater. Infected animals will discard viable pathogens directly into surface water (water fowl), or onto the ground surface (land animals). Rainfall events collect the infected feces from the watershed, and then either percolate into the groundwater or flow overland into surface water bodies (OECD, 2003).

Accordingly, the most common way to determine the risk of infection due to a water source is to measure the degree to which the water has been influenced by human and animal feces. Human and animal feces has been studied extensively, in order to determine which universal constituents might enable their influence to be measured (OECD, 2003). Such a constituent is referred to as an index parameter (OECD, 2003). In fact, even feces from healthy individuals invariably contain many types of organisms. Certain organisms are restricted to particular regions, or particular animal species (OECD, 2003). For example, the bacteria species *Enterococcus* is primarily found only in humans. Total coliforms, however, are bacteria that are always found in human, mammal, and bird feces. Certain coliforms are also found reproducing in the environment, so the presence of total coliforms does not guarantee fecal contamination (OECD, 2003). A subset of total coliforms is known as thermotolerant coliforms, often referred to as fecal coliforms. This provides a better, more specific indication of fecal contamination, but still some strands are found reproducing in the environment (OECD, 2003). *Escherichia coli* is the only thermotolerant coliform that has been found exclusively in fecal sources. In addition, *E. coli* is always present in both human and animal excreta, and outnumbers other thermotolerant coliforms (OECD, 2003).

Water is often tested for total coliforms, thermotolerant coliforms, and *E. coli*, in order to estimate the degree to which the water has been contaminated by fecal matter. Many testing methods are available. The most common methods involve culturing viable organisms. This can be done by either placing sample water in test tubes with appropriate nutrient solutions, or filtering sample through a membrane and soaking the membrane in a nutrient solution. The tube or membrane is cultured by incubation at an appropriate temperature (typically 35° C for total coliforms and 44.5° C for thermotolerant coliforms) for 24 or 48 hours. Viable organisms will multiply, producing gas in the test tubes and growing to form visible colonies on the membrane. The tube is a positive or negative test, which can be used in conjunction with a dilution series and multiple tubes at each dilution to calculate the “Most Probable Number” of organisms present. The membrane method provides a direct count of organisms, but usually also requires a dilution series in order to obtain a statistically reliable number of organisms to count.

When used for the purpose of measuring fecal contamination, coliforms and *E. coli* are referred to as index organisms (OECD, 2003). Indicator parameters, on the other hand, provide a measurement of the effectiveness of treatment processes. For example, if 10^4 *E. coli* per liter are detected before chlorination, and 10^1 are detected after, then chlorination removed 99.9% of the *E. coli*. This is referred to as a 3 log removal rate. One could suppose that 99.9% of the pathogens were also removed. However, many pathogenic viruses and protozoa are more resistant to disinfection than *E. coli*, and may not be removed to the same degree (OECD, 2003). Researchers have suggested that using a viral indicator might be better for measuring viral and protozoan disinfection. However, due to the simplicity and low cost of testing for coliforms and *E. coli*, these bacterial indicators are still widely used around the world (OECD, 2003). Caution must be taken when using them, to ensure that the treatment efficacy is not over-stated.

2.2.2 Turbidity

Turbidity is perhaps the most intuitive and simple measurement of water quality. Turbidity measures the degree to which suspended particles within the water scatter light (Droste, 1997). Above about 5 NTU, turbidity is visible to the naked eye, and higher turbidity is aesthetically displeasing. Turbidity itself is not a major health issue. There is nothing particularly dangerous about drinking clay particles. The health danger comes from the fact that pathogens are often attached to and protected by particulate matter, hindering disinfection. Highly turbid water also tends to clog filters, as large amounts of solids are removed from the water. This increases maintenance, sometimes to the point where a particular treatment strategy is no longer practical. As discussed in Section 2.3.1, household water treatment systems generally have an upper limit for turbidity, above which they no longer operate as designed. Unfortunately, that limit is often below the actual turbidity of drinking water supplies in developing countries (Collin, 2009; Stevenson, 2008).

Turbidity is related to the solids content of the water. Solids in water are classified as either dissolved or suspended. Dissolved solids are typically ionic particles which dissociate due to the polar nature of water. The molecular size of dissolved solids varies, but generally not more than a few orders of magnitude larger than the size of a water molecule (approximately 0.1 nm).

Dissolved solids form a homogeneous solution that is often visually undistinguishable from pure water. However, dissolved solids can contribute to the color of water (Droste, 1997).

Suspended solids are distinctly different from dissolved solids in that they are not incorporated into the liquid. They tend to be much larger, ranging from colloidal (approximately 1 nm to 100 nm) to fine (100 nm to 1000 nm) to coarse (>1000 nm). Colloidal particles are known for being very stable in solution, which means that under normal conditions and reasonable times they cannot be settled out. This stability in solution is due in part to their small size, and in part to their semi-polar nature, which allows them to form a charged barrier around themselves, hindering flocculation and settlement. Larger particles, such as clay (< 2 μm) and silt (2 to 50 μm), are less stable in solution and can be settled out if given enough time in a still environment (Droste, 1997; Coduto, 1999).

Turbidity can be measured by a variety of methods. In the laboratory, a turbidimeter can be used to pass light through a sample of water, measuring the quantity of light that is refracted from the sample at a 90° angle. The reading that is displayed is in Nephelometric Turbidity Units (NTU). The turbidimeter must be calibrated according to commercially available standards. Turbidity is a dynamic property, depending not only on the suspended particle size and concentration, but also on the exact configuration of particles within the sample. For this reason, turbidity generally varies by a few percent over the course of a single reading.

One common method for making field measurements is by using a Turbidity Tube (Stevensen, 2008; WHO, 2008). The tube is filled with sample water until a mark on the bottom of the tube is no longer visible to the naked eye. Within a treatment plant, one method is to place markers at various depths within a tank, which can be used to quickly read the approximate turbidity. Both of these methods become invalid below about 5 NTU, where the water appears clean to the naked eye. The calibration of the visual methods can be challenging for two reasons: the measurement depends on the subjective determination of exactly when the mark becomes invisible; and the physical property which is actually measured is slightly different from that measured in a turbidimeter. For these reasons, various calibrations have been proposed, which

attempt to correlate turbidity tube measurements (referred to as Turbidity Units, or TU) to NTU (Stevensen, 2008; Losleben, 2008). For the purpose of this report, NTU will be used exclusively.

2.2.3 Dissolved Oxygen

Slow sand filtration makes use of aerobic biological treatment processes (Manz, 2007). Accordingly, the Dissolved Oxygen (DO) content of the water is critical. Without DO, aerobic processes will cease, and may be replaced by undesirable anaerobic processes (Manz, 2007).

The DO concentration in water is limited by the saturation concentration, which depends on temperature (Chin, 2006). The saturation level decreases with increasing temperature. At 22°C (the temperature of the laboratory in this research), saturated DO is about 8.6 mg/L (Chin, 2006). At 35°C, saturation is about 6.9 mg/L.

2.3 Existing Viable Clean Water Alternatives

In recent years, in response to the Millennium Development Goals and the abundant needs of the developing world, there has been a lot of progress (United Nations, 2008). About 1.6 billion people have gained access to an improved water source since 1990 (United Nations, 2008). Of course, there are still many challenging situations, and nearly one billion people still lack access.

Experience has shown that the best and safest water management strategy is a multiple barriers approach: protect the source, destabilize and remove particles through multiple processes, disinfect, leave a residual disinfectant, protect the distribution system, pipe sufficient quantities to every point of use and teach users how to not re-contaminate water that is used for consumption (WHO, 2008). This system, combined with hygiene education and sanitation, is the best known method of eliminating water related diseases. However, such a comprehensive system requires years to design and a significant investment in infrastructure to build, as well as technical expertise to operate and maintain over the years. The per capita expense increases significantly if rural populations are considered. Many of the people who do not have access to improved water are rural. Such a system cannot be realistically expected to reach the one billion who need it for many years to come.

In the meantime, many low cost solutions are currently available (Stevenson, 2008). Depending on the source water quality, the funds available, climate, and local markets, many people use a variety of Household Water Treatment and Safe Storage (HWTS) technologies. These technologies are implemented in the home, directly at the point of use, minimizing the opportunity for re-contamination. They are often inexpensive enough that local businessmen can distribute the technology at a profit while poor people can still afford to buy it. They are generally simple enough to require minimal training, although insufficient training is often a cause for ineffective use. The disadvantages to HWTS frequently include lower treatment quality, low flow rates, and maintenance difficulties. However, for the one billion people without access to an improved source, HWTS can provide safer water, saving many lives and improving the lives of countless more.

2.3.1 Overview of HWTS technologies

Matthew Stevenson (2008) developed a framework for monitoring effective use of many types of HWTS technologies. His thesis offers a useful description of the various HWTS technologies available, including treatment capacity and limitations. All data in this section is based on Stevenson's work.

Chlorination – Sodium Hypochlorite Solution

This method consists of measuring one capful of a chlorine solution into a 20 L water jug of water. The user should wait for 30 minutes to allow adequate disinfection time. The chlorine solution is effective at disinfecting water, and costs are minimal at \$0.66 per person per year. While use is simple, complications arise when turbid water is used, as chlorine demand increases and disinfection byproducts proliferate. The WHO suggests that waters over 5 NTU are not suitable for chlorine treatment.

Chlorination – Aquatabs

Aquatabs are another form of chlorine disinfection with the advantage of using solid dry pills instead of a liquid product. However, the pills are slightly more expensive than chlorine solution. The manufacturer does not set an upper limit for turbidity; however, Stevenson's analysis

recommends an upper limit of 80 NTU. The WHO suggestion still seems to apply to aquatabs as well as to liquid chlorine.

SODIS – Solar Disinfection

Solar Disinfection, known as SODIS, is perhaps the least expensive and most surprising method available. SODIS makes use of the hot climate and direct sunlight available in most developing countries. Low turbidity water is collected in plastic polyethylene terephthalate bottles which must be less than 5 L and less than 10 cm thick. The bottle is then placed in direct sunlight, ideally on a corrugated roof, where the sun heats the water to above 50° C. The combination of temperature and UV-A light is lethal to many pathogenic organisms. Six hours of direct sunlight is required, although for simplicity most users are instructed to place the bottle on the roof for a full day. It is worth noting that many bacteria can multiply in these conditions, but most human pathogens are deactivated. Monitoring programs have reported diarrhea reductions of up to 40%, which is typical for successful water treatment systems. In order for SODIS to work, the water within the bottle must be less than 30 NTU; otherwise, insufficient UV-A radiation will penetrate through the water and pathogens will be shielded by the suspended solids.

Cloth Filters

Cloth filters were developed especially for treating cholera and guinea worm in areas where they are prevalent. Cholera and guinea worm have large carrier hosts. A cloth can be folded 4 to 8 times and have effective pore sizes of 20 µm, which is smaller than the size of the carrier host. Water is then filtered through the cloth, and large particles as well as cholera and guinea worm hosts are prohibited from passing through the cloth. While this is an effective technology even for highly turbid water, it only provides a barrier to cholera and guinea worm. The remainder of the waterborne pathogens and turbidity are allowed to pass through unhindered.

Ceramic Pot Filters

Ceramic filters consist of a large clay pot into which water is poured. The bottom of the pot is manufactured to contain pores ranging from 0.6 to 3 µm, which removes many bacteria and much of the suspended solids. A coating of colloidal silver is painted on the underside of the bottom of the pot. This coating serves to deactivate the bacteria that pass through the filter. A

safe storage container with a spout at the bottom is included in the unit, holding the pot in place. The primary disadvantage of this filter is that the flow rate is very slow and is further slowed down by turbid water. Many households find that this filter does not produce enough water. In addition, turbid water wears down the filter more quickly and requires frequent cleaning, which is burdensome on the user and increases the likelihood of cracks forming. Nevertheless, this filter provides a convenient all-inclusive system, which can be cheaply produced (\$6 to \$25 for a long lasting unit) by local trained potters and sold by local businessmen.

Biosand Water Filters

Biosand filters consist of either a plastic or concrete storage container which houses a layer of sand between 20 and 40 cm thick. Water is filtered through the sand and particulate matter (including organic matter) is removed by mechanical and biological processes. Biosand filters generally have much higher flow rates than ceramic filters. However, biosand filters require frequent cleaning if the source water has high turbidity, and cleaning decreases the treatment efficiency considerably. Generally, pretreatment is recommended if the source water has turbidity higher than 100 NTU. The cost of a biosand filter is generally slightly higher than a ceramic filter; however, the lifetime of the filter is significantly longer, depending primarily on whether the filter is plastic or concrete. Similar to the ceramic filter, biosand filters can be manufactured locally, using locally available materials.

2.3.2 Slow Sand Filtration

Slow sand filtration (SSF) has been used for nearly 200 years as an effective method for treating drinking water at the municipal scale (Droste, 1997). SSF consists of distributing water across the top of a sand bed, passing water through 0.5 to 1.5 meters of sand, and collecting the water at the bottom of the filter. Sand beds are generally quite large; the average area is about 100 to 200 square meters (Droste, 1997).

Particulate matter from the water is trapped on the sand grains according to several mechanical mechanisms, including hydrodynamic forces, diffusion, sedimentation, inertia, and interception (Droste, 1997). These occur throughout the depths of the sand bed. Within the first few weeks after commissioning a sand bed, the top two to five centimeters of sand form a microbial

zoogeleal film. This is essentially a food chain of microbial organisms which efficiently consumes and decomposes itself and any organic matter which passes through the filter. In addition, the largest inorganic particles are also blocked from entering the filter at the surface of the sand, decreasing pore sizes and proceeding to trap smaller and smaller particles. This crucial layer has been termed the *schmutzdecke*, which is German for “dirty layer”.

Since most of the particulate matter from the water is removed in the *schmutzdecke*, this layer tends to become clogged and increase head loss considerably, decreasing the flow rate through the filter. For this reason, SSF beds must be cleaned on a regular basis, averaging about once a month (Droste, 1997). The most common method of cleaning is by scraping off one to two centimeters of sand, which will later be washed off site and returned to the filter bed all at once, after a specified amount of sand has been removed. This process decreases treatment efficiency for one to two days, while the *schmutzdecke* ripens on the new surface layer (Droste, 1997).

In a SSF bed, the water layer above the sand can be one meter or more, and is generally constantly flowing (Droste, 1997). If the downward flow of water ever ceases for any length of time, then the biologically active *schmutzdecke* is at risk of becoming anaerobic (Manz, 2007). This can cause some of the organisms to become dormant, and others to begin anaerobic digestion. The result is that treatment efficiency decreases, and the effluent begins to exhibit aesthetically displeasing odors and tastes. After flow is resumed, some time is necessary for the *schmutzdecke* to return to normal (Manz, 2007).

2.3.3 Biosand Filters

The Biosand Water Filter (BSF) was first developed in the early 1990’s by Dr. David Manz, from the University of Calgary (Manz, 2007). Biosand filters make use of a simple yet important modification to SSF technology: the addition of a supernatant water layer engineered to permit intermittent flow. During the pause phase of the filter, the water level is maintained at five centimeters hydraulically, by placing the outlet at the same elevation as the desired water level. Five centimeters has been found to be thin enough that oxygen is able to diffuse through the water and reach the *schmutzdecke*, avoiding anoxic conditions (Manz, 2007). This modification

permits the use of SSF at the household scale, filtering as much or as little water as the family needs. As of 2007, over 200,000 BSF's have been installed around the world (Manz, 2007).

The treatment process of a BSF is exactly the same as that described for SSF above (Manz, 2007). The operation of a BSF is very simple. Water is poured in the top of the filter as frequently as needed. Once water is poured in the top, it will pass through the diffuser and accumulate on the supernatant layer, increasing the pressure on the top of the filter. The increased pressure will cause water that is already in the filter to pass through, flowing down the sand and up the pipe to the spout. Water is collected from the spout in a separate safe storage container. At this point, the water can either be chlorinated for redundant treatment, or used directly. Daily use will help to prevent anoxic conditions from occurring, improving the filtrate quality (Manz, 2007).

The flow rate in a BSF can be up to 0.6 L/min, which generally provides plenty of water for a typical household to use the filtrate for all drinking, cooking, cleaning, and hygienic purposes (Manz, 2007). As the filter is used and the *schmutzdecke* begins to clog, the flow rate will slow down. Once the flow rate is too slow for the user, cleaning must occur to increase the flow.

Regular cleaning of BSF's is generally conducted using a different procedure from SSF, because removing and returning sand at a household level inevitably involves the loss of some sand (Manz, 2007; Stevenson, 2008). Since the grain size in a BSF is carefully controlled, losing too much sand at a household level could cause the BSF to become ineffective or unused. For this reason, cleaning is conducted using a technique known as "Swirl and dump" (Stevensen, 2008). Further discussion is provided in Section 3.3.4.

2.4 Fabric Map of Kaolinite

In this research, the chemistry of the water was found to be critical for duplicating the highly stable high turbidity source waters. Palomino and Santamarina (2005) discuss the chemistry and behavior of kaolinite clay particles. This section is entirely based on Palomino and Santamarina (2005). Discussion of experimental observations related to the fabric map of kaolinite are

included in Section 4.1. In particular, pH and ionic concentration are found to control the dispersion of kaolinite clay.

Palomino and Santamarina (2005) present both theoretical and experimental observations regarding the dependence of kaolinite particle-particle interactions on pH and ionic concentration. Kaolinite particles consist of sheets with both aluminum and silica located at different parts of the sheet. These two elements attach OH^- or H^+ molecules at various pH. The ionic concentration of the fluid surrounding individual particles changes the effect of local charges on the clay.

Due to the effects of both pH and ionic concentration, at certain points particles will tend to attach end to end but the faces will remain repulsed. At other points, particles will attach face to face and quickly grow in size. At other points, the end of one particle will attach to the face of another. These changes in particle-particle interactions affect the manner in which particles conglomerate and flocculate, controlling the particle size distribution and density. The boundaries between these interactions do not appear to be sharply delineated, but rather gradually shift from one to the other. Figure 1 presents the concluding figure.

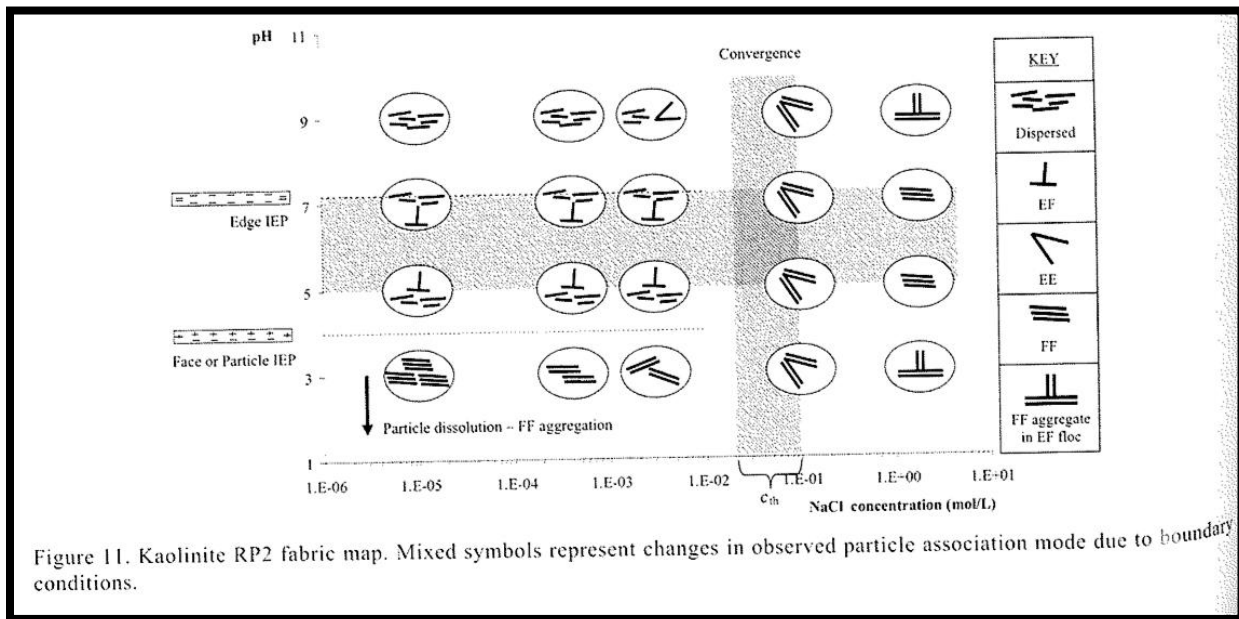


Figure 1 – Fabric Map of Kaolinite
(Palomino and Santamarina, 2005)

Although this is not suggested by Palomino and Santamarina, the same localized charge configuration (high pH and low ionic concentration) that disperses particles and keeps them suspended in solution will also likely have an effect on the tendency of suspended particles to interact with filter media. Therefore it is not simply the size of the particles, but also the chemistry which influences filter effectiveness.

3 Methodology

This laboratory based research project was divided into three modules. The first module was developing the feed water. Section 3.1 describes the process of developing target criteria, and iteratively testing many ideas for how to meet those criteria. Eventually, it was determined that if kaolinite clay is mechanically dispersed in basic conditions with a low ionic concentration, then the resulting turbidity will be highly stable. By controlling the concentration of kaolinite added, any desired turbidity can be reached and maintained. Based on these findings, a manufacturing process was developed which successfully and consistently attained the target properties.

The second module was designing and constructing bench-scale filter apparatuses, described in Section 3.2. Five filters were constructed using 4" diameter Plexiglas tubes. One filter was set as the control filter, designed to be a scaled-down version of one of the standard designs for the Biosand Water Filter (BSF) used around the world. The remaining four modified filters were constructed exactly the same as the control, except with the addition of a Dual Sand Layer (DSL) stand, which was fixed above the lower sand layer, to hold the DSL in place. All filters were equipped with a sample port immediately above the lower sand layer, a diffuser basin, a cover, and an adjustable effluent tube. This second module concluded with the commissioning and ripening of the filters, to prepare them for experimental procedures.

Module three consisted of operating (Section 3.3), observing (Section 3.4) and maintaining (Section 3.5) all five filters for about three months. The filters underwent three different configurations, iteratively designed to obtain specific data. The first phase examined the effect of DSL sand thicknesses of 1 cm to 5 cm, the presence of coarse sand beneath the fine sand, and increased daily flow rates. Based on the data from phase one, the second phase increased the sand thickness up to 9 cm, and looked at a thicker supernatant water layer. These first two phases were analyzed, and a set of recommendations were drafted. A third and final phase was conducted in order to confirm the efficacy and details of the recommendations.

3.1 Feed Water

Based on the results of previous research, it was determined from the beginning that developing feed water would be both critical and challenging (Collin, 2009). Field testing of BSF's have

measured turbidity reduction ranging from 20% to 98% for high turbidity feed waters (Stevenson, 2008; Kikkawa, 2008; Collin, 2009; see discussion in Appendix A). Collin’s laboratory testing of DSL BSF’s, on the other hand, obtained 98% turbidity reduction. This limited the applicability of the laboratory results to field conditions, since the feed water used in the laboratory was more easily treated than the water available in most field conditions.

One goal for this research project was to determine and establish a methodology for manufacturing feed water that would resemble critical field conditions, consisting of highly stable suspended particles that resist treatment to the greatest possible extent. The purpose was so that the results of this experiment would be applicable to the worst possible conditions in the field, such that less critical field conditions would result in better treatment than observed in the laboratory. This goal was successfully met, after many failed attempts. The failed attempts are described in Section 3.1.1, while Sections 3.1.2 and 3.1.3 describe the materials and process that was successful.

The target criteria for the feed water are described in Table 1. These were developed based on descriptions of water quality from several surface water supplies located in Northern Region, Ghana, as described in Appendix A.

Table 1 – Feed Water Target Criteria

Criteria	Value
Turbidity ¹	200 NTU
<i>E. coli</i>	400 CFU / 100 mL
Turbidity reduction after settling 24 hours	< 60 %
Turbidity reduction after Biosand Filtration	< 90 %

Note: 1) Target turbidity for the second half of phase 3 was switched to 1000 NTU.

In addition, in order to grow and maintain a ripe *schmutzdecke*, the water used as a base needed to have no residual chlorine concentration, and have a microbiological population typical of natural (untreated) waters.

3.1.1 Process Development

In developing the process for manufacturing feed water, many experiments were conducted and analyzed. The initial unsuccessful experiments are briefly described but are not fully presented. The final successful set of experiments and the associated observations are discussed here and more fully presented in Section 4.1.

The most straight-forward and logical approach to introducing turbidity is to add clay to water. This is what Collin (2009) did with Charles River water, which resulted in high turbidity reduction rates. Based on Collin's experience, other strategies were initially considered, only to return to a variation of the same strategy.

Initial Experiments

The initial idea was to attempt to duplicate the conditions that may occur in the seasonal rivers and reservoirs in Ghana. The first set of experiments began with adding sand and loam to laboratory grade water at different ratios of water to material. The material was added by scooping it into the water and then mixing with a stirring rod until the appearance was uniform. This was allowed to settle in a beaker for a period of one week, and was tested for turbidity at 24 hour intervals. Within seconds after the stirring ceased, the majority of the material settled on the bottom of the beaker. The water, however, remained highly turbid, in proportion to the amount of material added. However, within 1 to 2 days the turbidity due to sand decreased to less than 1 NTU, and that due to loam decreased to approximately 30% after one day, and 10% after one week.

Next it was attempted to sieve the sand and loam through various sieve sizes. In addition, a jar tester was used to provide consistent mixing for several minutes, and the beakers were allowed to settle for 1 hour prior to testing for initial turbidity. This experiment also compared results for laboratory grade water and Salisbury Pond water. It was interesting to observe that the laboratory grade water resulted in much higher turbidity.

Based on the above experiments, loam passed through a No. 50 sieve provided the most stable suspended particles, especially in laboratory grade water. However, this resulted in two

problems: first, laboratory grade water is not natural water, and will therefore not grow or maintain a ripe *schmutzdecke*; and second, the large amount of material left on the bottom of the container would need to be removed prior to pouring water through the filters. While this would occur naturally in a river or lake when water is decanted from the surface, in a laboratory setting this would be problematic.

The next series of experiments attempted to address the first of the above problems by strictly using Salisbury Pond water. The second problem was addressed with two strategies: first by filtering the water with loam through cloths; and second by using kaolinite clay instead of loam. The cloths proved very difficult to work with, and the Salisbury Pond water with loam did not hold the suspended particles in solution well enough. The clay was more successful in that much less material settled to the bottom immediately, but after one week of settling a 99% reduction in turbidity was observed.

Additionally, the loam-cloth strategy and the kaolinite clay strategy were tested by passing each through a full-scale Kanchan™ style BSF in the laboratory. In both cases, the turbidity of the filter effluent was less than 1% that of the influent.

Fabric Map of Kaolinite

At this point, the literature review revealed a paper which describes the “fabric map for kaolinite” (Palomino & Santamarina, 2005). This paper outlines the behavior of kaolinite clay particles at varying pH and ionic concentration (as measured by moles per liter of sodium chloride). The paper concludes, both theoretically and experimentally, that the most deflocculated condition exists at basic pH (>7.2) and low ionic concentration (<0.001 mol/L NaCl). More details on the paper are provided in Section 2.4.

Based on the map outlined by Palomino & Santamarina (2005), several experiments were conducted to see if a critical pH and ionic concentration would result in the highly stable suspended particles desired. It was found that combining a high pH, low ionic concentration, and a high-speed blender to disperse the kaolinite particles would result in the stability described in

Table 1. Based on these observations, the successful procedure described in Sections 3.1.2 and 3.1.3 was developed.

3.1.2 Materials

The following four materials were used in the daily manufacture of feed water. Wachusett Reservoir water was used as a base. The pH was adjusted with Sodium Hydroxide. Kaolinite clay was dispersed into the water using a blender, and wastewater from the Upper Blackstone Water Pollution Abatement district was used to provide fecal contamination.

Wachusett Reservoir

Based on the fabric map of kaolinite (discussed in Sections 2.4 and 3.1.1), a source of water with low ionic concentration was required for critical particle stability. According to Palomino & Santamarina (2005), kaolinite particles begin to aggregate between 0.001 and 0.01 mol/L NaCl, and gradually become more aggregated up until 1 mol/L NaCl. (Note that Palomino & Santamarina are using sodium chloride in lieu of ionic concentration. Ionic concentration actually depends on the weighted sum of all ions, with higher charge ions weighted by the square of the charge.) Therefore, an equivalent ionic concentration of approximately 60 mg/L NaCl or less was desired.

Due to the challenges associated with decreasing the ionic concentration of a water supply without sterilizing it, the strategy chosen was to find an existing pristine natural water supply. Two local ponds (Salisbury Pond and Indian Lake) were tested for sodium concentration, with the resulting estimated ionic concentration well over 250 and 160 mg/L as NaCl, respectively. This explained the kaolinite behavior observed in prior experiments with Salisbury Pond water. Therefore, these water bodies were ruled out as potential sources.

To find a more pristine and consistent water supply, the nearby Wachusett Reservoir was examined. Wachusett Reservoir is part of the metropolitan Boston water supply, and is known for being quite pristine. The ionic concentration of water collected from Wachusett Reservoir was tested on several occasions, and found to be consistently less than 50 mg/L as NaCl. For

further discussion on observations regarding ionic concentration and particle stability, see Section 4.1.2.

Over approximately four months, water was collected from several locations from Wachusett Reservoir. Water was collected in dedicated 5 gallon containers and refrigerated until 24 hours prior to use. Location 1 was on the north side of the basin where the Quinepoxet River (and Quabbin Transfer Pipe) enter the reservoir. This basin is contained by the railroad on the east side of the basin. Sampling occurred at the shore, where the water was shallow and relatively high turbidity existed. Sampling from location 2 was conducted from the railroad bridge where the basin from Location 1 enters the rest of the reservoir. Sampling occurred from the steps at the bridge abutment, where the water was well over five feet deep. Once this basin froze over by mid December, water was drawn once from the Cosgrove Intake structure (Location 3), where the Boston water supply is withdrawn. Due to security issues at the Cosgrove Intake, the fourth and final location was the Clinton Pump Station, located on the north side of the reservoir, where the town of Clinton withdraws their water supply. This location was used throughout January, February, and March, 2010. The final collection was conducted after the reservoir had thawed, so location 2 was used.

Sodium Hydroxide

According to Palomino & Santamarina (2005), the optimal pH is above 7.2. Therefore, before adding clay to the feed water, the first step was to adjust the pH. Sodium Hydroxide (NaOH) was used to raise the pH of the feed water. This was selected because the only bi-product was sodium, and the concentration of sodium added was negligible compared to the background concentration.

It was observed that the pH would drift significantly and consistently. The pH of the water would generally begin around approximately 6 to 6.7. At this point, drifting would occur in the upward direction. After stirring and reaching equilibrium, a pH of 6.6 to 6.8 was generally maintained. Then, NaOH would be added to raise the pH to approximately 8.1. At this point, drifting would reverse to the downward direction. By the time clay and wastewater were added, the final pH

was measured at around 7.2. For discussion on variations in pH and the stability of particles, see Section 4.1.1.

The most likely explanation for this pH drift is the carbon dioxide / carbonate system acting as a buffer. When water was collected and refrigerated, the system was at equilibrium with the atmosphere. Once the system is heated to room temperature (from 3.5°C to 22°C), the dissolved carbon dioxide solubility changes, resulting in a chain reaction which involves carbon dioxide, carbonic acid, bicarbonate, and carbonate. Since this reaction involves gas / liquid equilibrium, a slow pH drift which is accelerated by stirring is to be expected. (See Benjamin, 2002 chapter 7 for discussion)

Kaolinite Clay

Kaolinite clay was added to the feed water to introduce suspended particles, to raise the turbidity to 200 NTU for phases 1, 2, and the beginning of phase 3. The target turbidity was 1000 NTU for the second part of phase 3.

Kaolinite was selected as the particulate matter of choice for several reasons. First of all, the fabric map described in Sections 2.4 and 3.1.1 was available only for kaolinite. Second, kaolinite is known to be one of the most common clays in the world (Coduto, 1999), and is therefore likely to be causing high turbidity water in many locations around the world. A single uniform supply of kaolinite clay in excess of 10 kg was conveniently available in the laboratory, ensuring that sufficient quantities of the same material would be available for the duration of the research program.

Wastewater from UBWPAD

In order to duplicate fecal contamination of the feed water, wastewater was collected from the Upper Blackstone Water Pollution Abatement District, located in Millbury, MA. This wastewater treatment facility provides secondary treatment for the city of Worcester, MA and several surrounding communities (Upper Blackstone WPAD, 2009). Wastewater samples were collected from the primary clarifier effluent stream. Prior to this location, approximately 50% of

the solids are removed, and 30% of the organics, but no chemicals are added. Wastewater stock was collected approximately every two weeks, and refrigerated.

The target *E. coli* concentration was 400 colony forming units per hundred milliliters. Prior to commencing regular testing of *E. coli*, the stock concentration was estimated to be 100,000 CFU/100 mL based on an e-mail from the UBWPAD laboratory manager, Deborah LaVergne. Accordingly, for a typical 11 L daily feed water volume, 50 mL of stock wastewater was added.

Once regular testing of *E. coli* was begun, it was determined that immediately after collection the stock concentration was closer to 1,100,000 CFU/100 mL, and that the concentration decays over time. Therefore, beginning January 28, 5 mL of stock wastewater was added each day to 11.5 L of feed water.

3.1.3 Manufacture Process

Step 1

The necessary volume of Wachusett Reservoir water was measured, and brought to room temperature. This was normally conducted in pre-marked 5 gallon containers 24 hours in advance, to allow for thermal equilibrium to be established. In some cases, the water was heated on electric heaters to accelerate the process.

Prior to January 28, the daily volume was 11 L. From January 28 on, 11.5L was used, to allow for *E. coli* sampling of the feed water and extra water in case of spilling. Occasionally, greater volumes were required for the sake of filter maintenance or additional experiments. In these cases, the quantity of clay and wastewater stock were scaled proportionately.

Step 2

The initial pH of the Wachusett Reservoir water was measured under continuous stirring. As discussed in Section 3.1.2 – Sodium Hydroxide, the pH would consistently drift at this point.

Step 3

Sodium Hydroxide (NaOH) was added to raise the pH. As discussed in Section 3.1.2 – Sodium Hydroxide, the final target pH was above 7.2. In order to reach this level, experience indicated that the pH needed to be adjusted to above 8.0 at this point.

Prior to February 3, the NaOH stock solution was 0.1 N. From February 3 on, a stock solution of 1.0 N was used. The volume of NaOH stock required to reach the desired pH ranged from 0.3 mL to 0.7 mL of 1.0 N solution.

Step 4

At this point, kaolinite clay was measured and added to the feed water. The standard daily mass was 2.300g, with proportionately higher amounts on days where more feed water was required. This resulted in turbidity which averaged 197 NTU with a standard deviation of 6 NTU.

Clay was added using a blender conforming to the specifications for hydrometer analysis of soil particles, as described in ASTM D422 (Hamilton Beach Model No. 933). The blender cup was filled with 600 to 750 mL of the pH adjusted feed water, a portion of the pre-measured clay was scooped on top of the water, and the mixture was blended for a minimum of one minute. The contents of the cup were then poured in a dedicated 5 gallon container. This process was repeated five to ten times until all of the clay was used up. The last time, the cup, blender, and clay dish would be rinsed with pH adjusted feed water and blended to ensure that all of the clay made it into the feed water. Finally, the remaining pH adjusted feed water would be poured into the second 5 gallon container.

Step 5

Wastewater stock was measured using either graduated cylinders or micropipettes, and added to the turbid feed water. The standard volume was 50 mL prior to January 28, and 5 mL afterward, due to better data on the *E. coli* concentration of the wastewater stock solution.

Step 6

The final solution was vigorously shaken to ensure uniformity, and a 30 mL sample was taken for a turbidity measurement. The container was placed on a stirring plate and the pH was measured.

3.2 Laboratory Apparatuses

This section describes the design, construction, and configuration of the five laboratory scale filters. Section 3.2.1 describes the plastic materials used in constructing the filter apparatuses. Section 3.2.2 describes the filter media, fine sand, coarse sand, and gravel. Section 3.2.3 describes the installation procedure. Finally, Section 3.2.4 describes the configuration of the DSL for each phase.

These filters were modeled after the full-scale design of the KanchanTM BSF, as described in the thesis of Clair Collin (2009). The laboratory filters were constructed with a filter tube with inside diameter of 10.16 cm, compared to a diameter of 37.5 cm at the sand surface for the KanchanTM design. The area scale factor was 13.6. For discussion of the comparison of flow rates, see Section 3.4.1. The vertical scale used for the laboratory filters was 1:1.

3.2.1 Size and Design of Filter Apparatuses

This section lists all of the components of the filters except for the media, which is described in the following section. All dimensions are given in centimeters.

Filter Base (All Filters)

Material: Plexiglas
 Thickness: 0.64 cm
 Dimensions: 24.38 cm x 10.38 cm

Outer Filter Tube (All Filters)

Material: Plexiglas
 Outside Diameter: 10.80 cm
 Inside Diameter: 10.16 cm
 Height: 60.96 cm

DSL Stand Ring (All Except Control Filter)

Material: Plexiglas
 Outside Diameter: 9.53 cm
 Inside Diameter: 8.89 cm
 Height: 1.0 cm

DSL Stand Base (All Except Control Filter)

Material: Lexan
 Diameter: 9.53 cm
 Thickness: 0.32 cm
 Hole Diameter: 0.2 cm
 Hole Density: 11 / 81.07 cm²

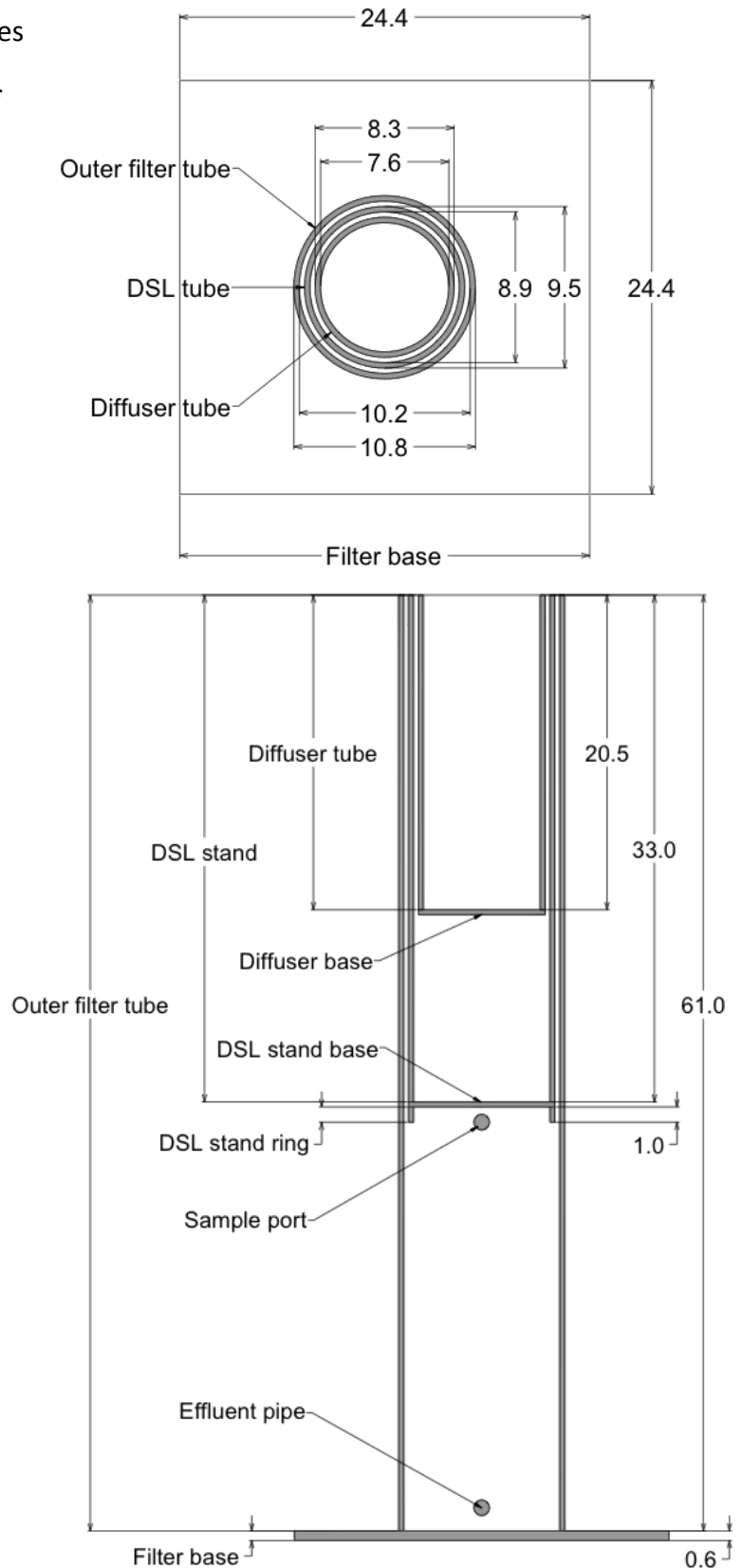


Figure 2 – Laboratory Filter Design

DSL Stand Tube (All Except Control Filter)

Material:	Plexiglas
Outside Diameter:	9.53 cm
Inside Diameter:	8.89 cm
Height:	33.0 cm

Diffuser Base (All Filters)

Material:	Lexan
Diameter:	8.26 cm
Thickness:	0.32 cm
Hole Diameter:	0.2 cm
Hole Density:	3 / 81.07 cm ² (filter 1 – no DSL)
Hole Density:	2 / 62.07 cm ² (filters 2 through 5)

Diffuser Tube (All Filters)

Material:	Plexiglas
Outside Diameter:	8.26 cm
Inside Diameter:	7.62 cm
Height:	20.5 cm

Filter Cover (All Filters)

Material:	Lexan
Diameter:	>10.2 cm

The outer filter tube was glued to the filter base using Plexitic plastic glue. The joint was sealed with epoxy to ensure a water-tight seal. The effluent pipe port and sample port were installed with a rigid tube connector which extended about 1 cm into the filter and enabled a rubber tube to be attached to the outside of the filter. The effluent tube was held in place by a system of sturdy wires, and the sample port tube was clamped closed until needed.

The DSL stand ring was placed on top of the lower sand layer. A slice was cut in it so that the sample port connector could pass through, enabling sampling to occur from the center of the intermediate water layer. The DSL stand base was glued to the DSL stand tube using Plextic plastic glue. The DSL stand base and tube were rested on the DSL stand ring. An additional support was created at the top of the DSL stand tube by taping a wire around the outside of the tube, and taping a second wire to the inside of the outer filter tube just below the first wire. The first wire rested on the second wire, providing adequate support for the DSL stand.

The diffuser base was glued to the diffuser tube using Plextic plastic glue. None of the diffusers leaked through the glued joint throughout the entire project. The diffuser for filters 2 through 5 were held in place using the same wire support system as for the DSL stand. One wire was taped to the outside of the diffuser tube, and a second wire was taped to the inside of the DSL stand just below the first wire. These wires were the sole support for the diffusers, and provided adequate support throughout the project.

Since filter 1 was not equipped with a DSL stand, the support for this filter was constructed using a Lexan circular piece similar to the filter covers. A hole approximately 7.6 cm in diameter was cut out of the piece, and the piece was glued onto the top of the diffuser. The piece was then rested on the top of the filter tube.

3.2.2 Filter Media

The filter media consisted of gravel, coarse sand, and fine sand.

Gravel

Gravel formed the base of the main filter bed. Gravel was passed through a ½ inch (12.7 mm) sieve and retained on a ¼ inch (6.4 mm) sieve. The gravel was washed thoroughly and dried prior to use in the filters. The depth of gravel was 5 cm for all filters.

Coarse Sand

Coarse sand was used to support the fine sand layer. The coarse sand was placed on the gravel in the main filter bed, and placed directly on the DSL Stand Base, for all filters except filter 1 in all phases and filter 2 in phase 1. Coarse sand was passed through a 5.66 mm sieve (US No. 3 ½) and retained on a 4.76 mm sieve (US No. 4). The coarse sand was washed thoroughly to remove fines, and dried before filter installation. The depth of the coarse sand in the main bed of the filter was 3.5 cm, and the depth in the DSL was 1 cm.

Fine Sand

Fine sand was passed through a 1 mm sieve (US No. 18) and washed thoroughly to remove the fines, and dried before filter installation. The depth of fine sand in the main filter bed was 18.5 cm. The depth of fine sand in the Dual Sand Layer varied according to the filter configuration, as described in Section 3.2.4.

3.2.3 Filter Installation

Filter installation was begun by adding 2.8 mL of stock sodium hypochlorite solution (5.65 – 6 % NaOCl) to 4 L of tap water. 800 mL of water was placed in each filter. The gravel and coarse sand were each placed in to the prescribed depth. Each layer was leveled prior to proceeding to the next layer. Care was taken at first to ensure that the layers did not mix. Then, 400 mL of tap water was added without chlorine addition. Fine sand was poured into the filter to a depth of 18 cm and leveled.

The final sand height and water height were measured. These values, combined with the filter dimensions and the known water volume were used to calculate the pore volume of the main filter bed. This pore ratio was used to calculate the additional pore volume contributed by the DSL for each configuration.

The filters were disinfected for 72 hours prior to flushing 3.5 L of Wachusett Reservoir water through, to remove the chlorine. The filters were then configured for phase 1 as described in the following section. Filters were operated using feed water 14 times in 33 calendar days in order to ripen the filters prior to commencement of phase 1.

3.2.4 Filter Configurations

Filter operation was divided into three phases. The filters were reconfigured for each phase. All filters were exactly the same from the base up to the top of the lower sand layer. The individual configurations consisted of whether a DSL was included (all except the control filter in each phase), whether a coarse sand layer was included, the thickness of the fine sand layer, the thickness of the supernatant water layer, and the volume of water poured through the filter each filter operation. Details on the configuration for each phase are presented in Table 3 through Table 5.

All three phases were used to test the effects of adding a DSL to the BSF design. The first two phases tested specific parameters, including the presence of a coarse sand layer supporting the DSL; DSL fine sand thicknesses of 1 cm, 3 cm, 5 cm, 7 cm, and 9 cm; supernatant water layers of 1 cm and 3 cm; and operational volumes of approximately 200% and 300% of the pore volume. All phase 1 and 2 tests were conducted with feed water at 200 NTU.

Phase 3 was designed to test and refine the recommendations presented in chapter 5. Accordingly, phase 3 focused on decreasing the operational volume to less than 100% of the pore volume, and testing the filter reactions to shorter pause times and turbidity spikes of 1000 NTU. In addition, the supernatant water layer of some filters was increased to 5 cm.



Figure 3 – Laboratory Filters

Table 2 – Filter Label Code

Expression	Example	Meaning
#a/b/c	1a	filter number and phase
Control	Control	control filter – no DSL modification
#cm	3cm	depth of DSL fine sand layer in cm
#+#	7+3	depth of fine sand in cm + depth of supernatant water in cm
#L	3L	daily water volume in L (default is 2 L or 2.25 L if not specified)

Table 3 – Laboratory Filter Configurations for Phase 1: Jan 4 – Feb 23

Filter No.	Pore Volume	Residence Volume	Intermediate Water	Coarse Sand	Fine Sand	Supernatant Water	Operation Volume
1a Control	1.02 L	1.43 L	N/A	0 cm	0 cm	5 cm	2.00 L
2a 1cm	1.05 L	1.20 L	1 cm	0 cm	1 cm	1 cm	2.00 L
3a 3cm	1.14 L	1.32 L	1 cm	1 cm	3 cm	1 cm	2.00 L
4a 3cm 3L	1.14 L	1.32 L	1 cm	1 cm	3 cm	1 cm	3.00 L
5a 5cm	1.19 L	1.40 L	1 cm	1 cm	5 cm	1 cm	2.00 L

Table 4 – Laboratory Filter Configurations for Phase 2: Feb 24 – Mar 22

Filter No.	Pore Volume	Residence Volume	Intermediate Water	Coarse Sand	Fine Sand	Supernatant Water	Operation Volume
1b Control	1.02 L	1.43 L	N/A	0 cm	0 cm	5 cm	2.00 L
2b 7cm	1.25 L	1.48 L	1 cm	1 cm	7 cm	1 cm	2.25 L
3b 7+3	1.25 L	1.62 L	1 cm	1 cm	7 cm	3 cm	2.25 L
4b 9cm	1.31 L	1.55 L	1 cm	1 cm	9 cm	1 cm	2.25 L
5b 5cm	1.19 L	1.40 L	1 cm	1 cm	5 cm	1 cm	2.25 L

Table 5 – Laboratory Filter Configurations for Phase 3: Mar 23 – Apr 7

Filter No.	Pore Volume	Residence Volume	Intermediate Water	Coarse Sand	Fine Sand	Supernatant Water	Operation Volume
1c Control 1L	1.02 L	1.43 L	N/A	0 cm	0 cm	5 cm	1.00 L
2c 7cm 1L	1.25 L	1.48 L	1 cm	1 cm	7 cm	1 cm	1.00 L
3c 7+5 2L	1.25 L	1.76 L	1 cm	1 cm	7 cm	5 cm	2.25 L
4c 9cm 1L	1.31 L	1.55 L	1 cm	1 cm	9 cm	1 cm	1.00 L
5c 5+5 1L	1.19 L	1.69 L	1 cm	1 cm	5 cm	5 cm	1.00 L

In most cases, reconfiguration only involved changing the height of the effluent valve in order to increase the supernatant water layer, or changing the volume of water poured through the filter each day. The only exceptions were filters 2, 3, and 4 during reconfiguration from phase 1 to phase 2. The DSL stand for these filters was removed and emptied. The lower sand layer was maintained prior to replacing the stand, and reinstalling the DSL according to the prescribed configuration.

3.3 Filter Operation

This section describes the experimental operation of the filters. Operation was conducted in three phases, each with different configurations and operational procedures. The configuration of each is described in Section 3.2.4, while the details for the operational procedures are described in Section 3.3.1, along with an overview of the three phases. Section 3.3.2 includes a narrative describing the daily operation. The final section provides the rationale behind testing the parameters at the timing chosen.

3.3.1 Overview of Experimental Phases

Filter operation was divided into three phases. The physical configurations of the phases are described in Section 3.2.4, while this subsection describes the test parameters and timings used for each of the phases. An in-depth description of the methodology and justification for the timing of tests, are included in the following subsections.

Table 6 describes the start and end dates of each phase, including the ripening phase.

Table 6 – Phase Overview

Phase	Start Date	End Date	Calendar Days	Operation Cycles
Ripening	12/2/09	1/3/10	33	14
Phase 1	1/4/10	2/23/10	51	39
Phase 2	2/24/10	3/22/10	27	19
Phase 3	3/23/10	4/7/10	12	10

Table 7 describes each filter event that impacted the operation of any or all filters except for filter maintenance. Maintenance of all filters is recorded in Table 8. Note that after the initial

setup, all filters were maintained solely due to flow rate except for during the transition from phase 1 to phase 2, when filters 2, 3, and 4 were maintained in the process of replacing the thin DSL with a thicker DSL.

Table 7 – Calendar of Major Filter Events

Date	Filter No.	Event Description
12/2/09	All	First day of filter operation – ripening period
12/7/09	All	Filter Setup – configured for Phase 1
12/7/09	All	Initial Filter Maintenance
1/4/10	All	First day of turbidity data
1/15/10	All	First day of reliable DO data
1/26/10	All	First <i>E. coli</i> test. Concentration 10x too high
2/24/10	All	Reconfigured for Phase 2
2/26/10	Filter 2	Sample port clamp broke Drained to lower sand layer until fixed 2/27
3/23/10	All	Reconfigured for Phase 3
3/27/10	All	Beginning of 1000 NTU feed water
4/7/10	All	Last day of filter operation

Table 8 – Maintenance Calendar

Date	Filter No.	Maintenance Cause
12/7/09	All	Initial Filter Setup
1/26/10	5	Flow Rate
1/27/10	4	Flow Rate
2/8/10	3	Flow Rate
2/17/10	5	Flow Rate
2/24/10	2, 3, 4	Filter Reconfiguration
3/17/10	1, 5	Flow Rate

Each filter was operated five to seven days per week. Daily filter operation consisted of pouring the daily volume of water through the filter and testing for the parameters listed in

Table 9 for phase 1, Table 10 for phase 2, and Table 11 for phase 3. The filtrate volume refers to the volume of filtrate that was collected prior to recording the time or beginning to take a sample.

Table 9 – Daily Filter Testing Parameters for Phase 1

Filter Numbers	Parameter	Location	Filtrate Volume
All Filters	Dissolved Oxygen	Port	Pause Time
All Filters	Stopwatch Time	Effluent	0.00 L
All Filters	Dissolved Oxygen	Effluent	0.75 L
All Filters	Stopwatch Time	Effluent	1.00 L
All Filters	Turbidity	Effluent	1.50 L
All Filters	<i>E. coli</i>	Effluent	1.53 L

Table 10 – Daily Filter Testing Parameters for Phase 2

Filter Numbers	Parameter	Location	Filtrate Volume
All Filters	Dissolved Oxygen	Port	Pause Time
All Filters	Stopwatch Time	Effluent	0.00 L
All Filters	Dissolved Oxygen	Effluent	0.75 L
All Filters	Stopwatch Time	Effluent	1.00 L
Control Filter	Turbidity	Effluent	1.50 L
DSL Filters	Turbidity	Effluent	1.75 L
Control Filter	<i>E. coli</i>	Effluent	1.53 L
DSL Filters	<i>E. coli</i>	Effluent	1.78 L

Table 11 – Daily Filter Testing Parameters for Phase 3

Filter Numbers	Parameter	Location	Filtrate Volume
All Filters	Dissolved Oxygen	Port	Pause Time
All Filters	Time	Effluent	0.00 L
Filters 1, 2, 4, 5	Time	Effluent	0.50 L
Filters 1, 2, 4, 5	<i>E. coli</i>	Effluent	0.50 L
Filters 1, 2, 4, 5	Turbidity	Effluent	0.70 L
All Filters	Dissolved Oxygen	Effluent	0.75 L
Filter 3	Time	Effluent	1.00 L
Filter 3	Turbidity	Effluent	1.75 L
Filter 3	<i>E. coli</i>	Effluent	1.78 L

3.3.2 Description of Daily Operation

Daily operation was primarily the same for phases 1 and 2, but changed significantly for phase 3. The only operational differences between the first two phases was that the volume of water poured through filters 2b, 3b, 4b, and 5b were 2.25 L instead of 2.00 L, slightly changing the timing of tests as described in Table 9 and Table 10. The following narrative describes daily operation of both phases 1 and 2. Changes for the daily operation of phase 3 are described subsequently.

Phases 1 and 2

Prior to pouring feed water into the top of the filter, the dissolved oxygen concentration was measured immediately above the lower sand layer, by withdrawing a sample of approximately 35 mL from the sample port into a 50 mL glass vial. DO was measured as described in Section 3.4.3. After measuring the DO, the withdrawn water was poured back into the top of the filter, to maintain the water level.

The feed water container was shaken vigorously, and 1 L was measured into a graduated cylinder. This was then poured into the top of the filter, and a stop watch was used to record the starting time. The first liter fit entirely within the diffuser basin. If more than 1 minute had passed since the feed water container was shaken, it was shaken again. A second liter of feed water was measured in the graduated cylinder, and the filter was topped off periodically until the entire second liter was poured in. The amount of time that was required to completely pour in the second liter depended first on the total depth of DSL sand and supernatant water, and then on the flow rate through the filter. This varied from less than a minute for filter 1 (the control) to several minutes for filter 4b, with the thickest sand layer.

The effluent water was collected in a 5 L pitcher, which was marked at 0.5 L increments. When the effluent volume reached 0.75 L, approximately 35 mL of water was poured into the pitcher, and the next 35 mL of effluent was collected in a vial. The collection was taken at an angle, to minimize bubbling and splashing, which would introduce further dissolved oxygen into the effluent. The contents of the vial were immediately tested for DO, as described in Section 3.4.3.

The purpose of pouring 35 mL into the pitcher was to ensure an accurate measurement of the cumulative volume by replacing the volume that was removed for DO testing.

When the effluent volume reached 1.00 L, the stopwatch time was recorded. This was used to calculate the flow rate as described in Section 3.4.1.

Once the filtrate volume reached 1 L, for filters with daily volumes of 2.25 L (2b through 5b), 0.25 L was measured in a graduated cylinder and poured into the filter. For filter 4a with a daily volume of 3 L, the third liter was not poured into the filter until after all samples had been taken.

Next, the turbidity sample was taken at the appropriate volume (1.50 L or 1.75 L), as described in Section 3.4.2. This was capped and stored until all filters were complete, and all turbidity readings were measured at once.

On days when *E. coli* was tested, approximately 100 mL of effluent was collected in a sterilized nalgene bottle immediately after the turbidity sample was taken. In addition, a sample was taken from the raw feed water. All of the *E. coli* samples were tested immediately following the filter operation of all filters, as described in Section 3.4.4.

This procedure was repeated for all filters in a staggered pattern each day. Filter 1 was generally operated first, followed by filter 2, etc..

Phase 3

Phase 3 differed from phases 1 and 2 in that only 1 L was poured through filters 1c, 2c, 4c, and 5c. Accordingly, flow rate was calculated based on only 0.50 L instead of 1.00 L, and turbidity and *E. coli* tests were conducted at 0.70 L and 0.50 L of effluent, respectively.

The operational procedure for filter 3c was the same as that described above for phases 1 and 2, since filter 3c was operated using 2.25 L.

3.3.3 Justification for Timing of Tests

The water quality of the filtrate varies significantly in a single filter operation. Before selecting a “timing” (filtrate water volume) for taking regular DO and turbidity measurements, several profiles of the filtrate were conducted. The results of these profiles were important to the recommendations in chapter 5 and are accordingly presented in Section 4.2.4 and discussed in the appropriate Targeted Observations of Section 4.3.

Based on the DO profile, it was found that the lowest DO was generally at 0.75 L. Note that this was not refined further than +/- 0.25 L. This timing corresponds as closely as possible to the point at which water that abode in the pore spaces of the *schmutzdecke* is expected to be exiting the filter. Although some mixing is clearly occurring (Figure 34), the filtrate DO at 0.75 L was consistently lower than the sample port DO, especially at longer pause times. This was the reason used for testing all filtrate DO measurements at 0.75 L. The minimum DO able to be measured was desired, in order to ensure that anaerobic conditions did not occur at any point within the filter.

Based on the turbidity profiles, the highest turbidity recorded was approximately when 1.5 pore volumes had passed through the filter (see Figure 34). In order to ensure consistent day-to-day testing, the pore volume of all filters in phase 1 and filter 1 in phase 2 was rounded to 1 L, and the turbidity tests were conducted at 1.5 L. Since the pore volumes (PV) of filters 2b, 3b, 4b, and 5b were significantly higher than 1 L, turbidity was tested at 1.75 L instead, and the flow volume was increased appropriately in order to ensure that the decrease in flow rate did not interfere with the peak turbidity measurement. *E. coli* samples were collected as close as possible to the turbidity samples so that both treatment efficiency metrics would measure the same quality water.

During phase 3, it was no longer desired to obtain peak turbidities, but rather, further clarification of the filter performance at less than 1 PV was desired. Turbidity was therefore tested at 0.7 L, and the changes in pore volume between filters were compared. Filter 3c was kept at the same testing procedure from phases 1 and 2, to ensure that potential changes in the feed water would not go unnoticed.

3.4 Testing Procedures

The primary laboratory tests conducted on a daily basis were flow rate, turbidity, *E. coli*, dissolved oxygen, and pH. The test methods for each parameter are described below.

3.4.1 Flow Rate

Flow rate was measured using an average flow over a significant portion of the filter run cycle. For filters with more than 1 L cycle volume (all filters in phases 1 and 2, and filter 3c), each filter was provided a dedicated 5 L receiving vessel, which was always emptied at the beginning of each cycle. A stopwatch was used to record the minute and second at which water was poured into the top of the filter, called the start time or t_s . The time at which the meniscus of the water in the receiving vessel reached the 1 L mark was recorded and called t_{1L} .

For filters with 1 L cycle volume, the effluent was collected and measured in a graduated cylinder. The start time was recorded along with the time at which 0.5 L of filtrate had been collected. From these data, the flow rates liters per minute were calculated. (See Section 3.3 for a complete description of the run cycle).

Although the 5 L receiving vessels were not volumetric glassware, the accuracy observed was sufficient for this purpose. The volume of water measured by the 1 L mark was accurate to within about 1%. In addition, the clarity of determining t_{1L} was generally accurate to within +/- 3 second. In some cases, low flow rates caused the water to drip rather than pour, causing waves which made measurement more difficult. In this case, accuracy was within +/- 10 seconds.

This method measures the average flow rate over a long period of time. Another alternative would have been to measure the time at which a small volume of water, such as 10 mL, passed through the filter at a certain point. However, a short range flow measurement would be highly variable depending on the rate at which water is poured into the filter, the relative head levels at any given time, and the rapid dynamic nature of the filter flow. (Flow first passes through the diffuser, then the upper sand layer, then the lower sand layer, with storage available at each stage, each at different rates and with different head losses depending on the instantaneous head above and below.) This would make it difficult to compare the flow rate from day to day. Using

a long term measurement provides consistency for monitoring purposes. In addition, a long term measurement is more appropriate to the needs of users in the field.

3.4.2 Turbidity

Turbidity was measured for the feed water prior to operating filters each day of operation. Turbidity of the filtrate was also measured for each filter at the designated point in the run cycle, after the volume of filtrate described in Table 12 had been collected from the filter.

Table 12 – Turbidity Test Timing

Filter Description	Filtrate Volume
Phase 1 all filters	1.50 L
Phase 2 filter 1a	1.50 L
Phase 2 all other filters	1.75 L
Phase 3 filter 3c	1.75 L
Phase 3 all other filters	0.70 L

Turbidity was measured using a HACH 2100N turbidimeter. The turbidimeter was calibrated once at the beginning of experimental procedures. It was observed that the change in calibration was negligible, even though the turbidimeter had not been calibrated in at least 1 year. Therefore, the turbidimeter was not recalibrated.

A 30 mL turbidimeter vial was filled to the mark with sample. The vial was carefully wiped clean with a lint free tissue. The turbidimeter was set to give a 10 second continuously updating average result, which helped to stabilize the reading. Immediately after placing the 30 mL vial into the appropriate slot and closing the cover, the 10 second averaging was reset, to avoid averaging with values measured while the cover was open.

3.4.3 Dissolved Oxygen

Dissolved Oxygen (DO) was measured daily for each filter twice. The first daily measurement was taken from the sample port just prior to filter operation. The second was taken from the effluent after 0.75 L had passed through the filter.

DO was measured using a Thermo Electron Corporation Orion 3 Star Benchtop DO Meter. Initial DO testing revealed that significant interferences were occurring when calibrated with water-saturated air, which is the normal calibration method recommended in the manuals for the meter and probe. Using the air-saturated water method, it was observed that calibrating the DO meter in air-saturated Wachusett Reservoir water prior to adjusting the pH and adding kaolinite, and then bubbling air through for 30 minutes to re-saturate would result in measurements significantly higher than saturation. Furthermore, the meter was found to be sensitive to the flow rate over the probe, which depended on the stirring rate and the size and shape of the container.

Stirring patterns were controlled by controlling all possible parameters. Calibration and all measurements were conducted in the same 50 mL vials in order to control all variables. A dedicated reliable low-speed stirring plate was maintained at the same mixing rate for all DO measurements. The BOD sleeve for the DO probe was used, with a built-in stirring rod at a consistent location. A dedicated burette stand was used to hold the DO probe at the same height for all measurements.

The most reliable calibration method was found to be air-saturated water using filtrate from the previous day's operation. The filtrate of the final DO measurement of the day would be kept in a DO vial. At least 30 minutes prior to operation, the sample would be bubbled with air to ensure saturation. The probe would then be calibrated daily just prior to measuring the DO of samples from the sample ports.

Probe maintenance was conducted whenever necessary. Maintenance consisted of replacing the electrode solution within the DO probe. Maintenance was determined to be necessary whenever the meter displayed an error indicating such. It was observed that after maintenance, the reading would drop consistently each day by about 3.5%. This was corrected by calibration, until the calibration was too low for the meter to function accurately.

After calibration, DO samples were collected and measured as follows:

1. Samples were always collected in a standard 50 mL vial.
2. For filtrate samples, the previous contents of the vial (~35 mL) were poured into the receiving vessel to replace the 35 mL that would be removed by collecting a DO sample. This was done in order to not interfere with flow rate measurements.
3. Samples were collected by tilting the vial, so that water flowed down the side of the vial, eliminating air bubbles and splashing, which could increase DO.
4. Vials were filled to approximately 35-40 mL.
5. The vial was immediately placed on the stir plate, and the DO probe was placed in vial. Approximately 1-5 mL would spill out of the top of the vial when the probe was set in place.
6. The DO meter was set to read continuously. Upon placing the probe to the vial, the measured DO would peak (from being in air) and then settle down at a stable reading. This stable reading was recorded as the DO. Although this reading was recorded as soon as possible, it was observed that the same reading +/- 0.03 mg/L would hold stable for several minutes.

3.4.4 *Escherichia coli*

E. coli was measured as an index organism for treatment efficiency of pathogens, as discussed in Section 2.2.1. *E. coli* measurements were made periodically, with special emphasis for before and after maintenance operations.

EPA Method 1603 was used, with modified membrane-Thermotolerant *Escherichia coli* Agar (modified mTEC agar). For details regarding agar preparation, petri dish preparation and the phosphate buffered rinse solution, see the EPA published method. Samples were tested using three volumes of each sample: 50 mL, 15 mL, and 5 mL.

Samples were filtered through a 0.45µm filter and rinsed with the rinse solution. The filter was aseptically transferred to a sterile petri dish with agar. All the dishes were collected and placed upside-down in a watertight bag in a water bath and incubated at 35°C for 2 hours. After 2 hours, the bath was switched to 44.5°C and incubated for 22 to 24 hours. After incubation was

complete, the plates were removed and the magenta or red colonies were counted on all filters. If any of the dishes had between 20 and 80 colonies (inclusive), only these dishes were used for calculating *E. coli* concentration. If none had between 20 and 80, then all dishes were used for each sample.

The equation used for calculating *E. coli* concentration for samples with only one dish with 20 to 80 colonies is:

$$\frac{N_{Colonies}(CFU)}{V_{Sample}(mL)} \cdot \frac{100mL}{100mL}$$

Where $N_{Colonies}$ is the number of colonies counted on the dish in units of Colony Forming Units, and V_{Sample} is the volume of the sample filtered through the dish in mL. The result is reported in units of CFU per 100 mL. For samples where multiple dishes had to be counted, the equation used is:

$$\frac{N_{Colonies_1} + N_{Colonies_2} + N_{Colonies_3}}{V_{Sample_1} + V_{Sample_2} + V_{Sample_3}} \cdot \frac{100mL}{100mL}$$

3.4.5 pH

The pH was measured and recorded three times on each filter operation day. First, the pH of the unmodified stock Wachusett Reservoir water was measured. This was used to determine the pH adjusting NaOH dose. After addition of NaOH, the pH was confirmed. Finally, after kaolinite and wastewater were added the pH was measured once more before operating the filters.

All pH values were measured using an Accumet Basic AB15 pH Meter, which is distributed by Fisher Scientific. The pH meter was calibrated periodically using three pH standards at 4.00, 7.00, and 10.00.

As mentioned in Section 3.1.3, the pH of the feed water was observed to consistently drift during the manufacturing of feed water. However, during calibration and whenever the calibration would be checked using a standard, the meter would quickly reach the expected value. Very little drifting of the meter itself was observed. Therefore it was concluded that the pH meter was

giving reliable values, and that the feed water pH was actually drifting due to reactions within the solution.

3.5 Filter Maintenance Criteria and Procedure

Filter maintenance was an important part of this research project. Filter maintenance on a BSF is only to be conducted when insufficient flow rate is observed. Determining the frequency of maintenance and the impacts of maintenance on treatment efficiency were critical in analyzing whether the DSL was beneficial, as discussed in Section 4.3.4. In order to obtain reliable results, a consistent procedure was used to determine when filter maintenance was necessary, and how to perform maintenance when necessary. These are described in this section.

Flow Rate Criteria

Determining when to conduct filter maintenance involved several considerations. First, the minimum flow necessary depends on the household daily water demand, which varies from household to household. Second, the flow rate observed in the laboratory is significantly different from the design flow rate of full-scale filters. These factors were considered against the intent to maintain the filters multiple times, so that the desired observations could be made. The minimum flow rate criteria was set at 0.11 L/min.

The critical flow rate is best user-determined in the field. The WHO Guidelines for Drinking-water Quality discuss the quantity of water by breaking it down into four categories of level of service (WHO 2008). Water use ranges from “optimal” at more than 100 L per capita per day, to “no access” at 5 L per capita per day. The main difference between these levels is the travel distance necessary for obtaining water, ranging from multiple household taps to greater than 30 minute travel to the nearest water supply.

The WHO estimates that 7.5 L is necessary for hydration and incorporation into food, and greater volumes are necessary for hydration. A “basic” level of service is estimated to provide 20 L per capita per day, and is expected for water sources that are not within the property but are less than a 30 minute walk away. It is expected that if a water tap is available on the property, the

BSF will not be necessary. The conditions expected for BSF implementation are therefore associated with approximately 20 L per capita per day.

Since the BSF is a household filter, the number of people in the household also greatly impacts the volume of water necessary in a day. For the purpose of this laboratory research, design volume of 90 L per day was selected. It was also assumed that the filter would not be in constant use. A conservative estimate of 8 hours per day was used to determine the maintenance criteria. If the recommendations in chapter 5 are followed, filter maintenance won't actually be conducted until the filter is operating approximately 20 hours per day. Using 90 L in 8 hours for a full-scale filter, the following flow rate per unit area was calculated as the full-scale minimum flow rate criteria for maintenance:

$$\frac{90L}{480\text{min} \cdot 0.126m^2} = 1.49 \frac{L}{\text{min} \cdot m^2}$$

Table 13 presents the flow rates and cross-sectional areas for several common BSF designs, and compares these with the initial flow rate observed in the laboratory and the selected minimum flow rate criteria.

Table 13 – Flow Rates for Laboratory-Scale and Full-Scale BSF's

	Area (m ²)	Flow Rate		
		L/min	L/min/m ²	Gal/min/SF
CAWST	0.123	0.60	4.9	0.120
HydrAid TM	0.126	0.78	6.2	0.153
Kanchan TM	0.33	0.11	3.0	0.074
Laboratory Initial	0.00811	0.23	28.4	0.696
Laboratory Minimum	0.00811	0.11	13.6	0.333

Note: Design flow rates and filter dimensions for CAWST, HydrAidTM, and KanchanTM designs collected by Collin (2009).

The laboratory configuration was modeled after the KanchanTM filter, used by Collin (2009) and Kikkawa (2008). In spite of the fact that the media is configuration is the same, the initial flow

per unit area observed in the laboratory is approximately ten times the design flow rate for the KanchanTM filter. This is due to several factors.

First of all, this does not appear to be due to short-circuiting or any filter malfunction, since the initial flow rate was consistent among the five laboratory filters. Therefore it must be due to changes in the filter design. For the full-scale filter, much of the head loss occurs at the sub-drain, before entering the riser pipe. In the KanchanTM design, there are only two small holes in the plastic through which all of the water must flow (based on physical observation by the author). This could be the critical factor in controlling the overall flow rate, decreasing it by a significant factor. In the laboratory filters, this limitation is removed, increasing the overall flow rate.

Another contributing factor is the increase in freeboard within the laboratory filter design. To accommodate modifications, the columns were designed to be about 14 cm taller than the KanchanTM filter, nearly doubling the maximum head. This significantly increases the flow rate, especially for the first liter that passes through the filter, while the water level is being continually topped off.

The critical flow rate was initially set based on these observations, prior to the first filter maintenance. After the first two weeks of phase 1, all of the laboratory filters generally operated at flow rates below about 0.17 L/min, even after maintenance. This would convert to 21.0 L/min/m², which is much closer although still significantly above the full-scale design flow rates in Table 13.

In order to determine the laboratory scale minimum flow rate, the full-scale minimum flow rate criteria calculated above as 1.49 L/min/m² was used as a starting point. The above observations regarding comparison between the full-scale and laboratory-scale designs were considered. It was also observed that 1.49 is approximately half of the design flow rate for the KanchanTM filter, which was used as the basis for the laboratory designs.

The full-scale minimum flow rate per unit area of 1.49 L/min/m² was multiplied by a factor of 9, to account for the change in freeboard head, and the removal of under-drain head loss. This was then converted to L/min in the laboratory filter and rounded to 0.11 L/min. It was observed that this is approximately 50% of the initial flow rate. 0.11 L/min was used as the minimum flow rate, beneath which filter maintenance was conducted.

Filter Cleaning Method

When the flow rate criteria triggered filter maintenance, the following procedure was used. A stirring rod with a tip cut to 1 cm was used to ensure that the same depth of 1 cm was always reached.



Figure 4 – Filter Maintenance Rod

1. The sand was stirred with the tip fully inserted into the sand.
2. The supernatant water was decanted and discarded.
3. Feed water was placed into the filter carefully so as not to overly disturb the sand.
4. Steps 1 through 3 were repeated until the turbidity of the stirred water was visually comparable to that of the feed water.

4 Results and Discussion

In this chapter, the results and associated observations of this research program are presented for two categories: the first focused specifically on the feed water (Section 4.1), and the second focused on the filters (Sections 4.2 and 4.3).

Section 4.1 describes two key observations concerning the stability of kaolinite clay suspended particles, both of which are consistent with the fabric map of kaolinite described in Section 2.4 and experimental observations. Specifically, suspended kaolinite particles are observed to be more stable in suspension at slightly basic pH and at low ionic concentration. These conclusions were used to successfully duplicate stable high turbidity conditions, providing a basis for filter operation experiments.

Filter operation consisted of a ripening phase and three operational phases which were each characterized by a unique set of filter configurations and operational schemes. The results of each of these phases are presented graphically in Section 4.2 in terms of flow rates, turbidity, *E. coli*, and DO.

Section 4.3 draws out several targeted observations based on the results presented in Section 4.2. Although the presence of the Dual Sand Layer (DSL) is found to affect DO, the most important parameters for water quality are shown to be the pause time, the water volume, and especially the pore volume.

Also described in Section 4.3, the addition of a DSL was found to have negligible benefit under high stability and high turbidity conditions, as modeled in the laboratory. Some field conditions with low particle stability are expected, under which the DSL may be more beneficial.

The benefits of the DSL for high turbidity and high stability as modified in the laboratory are addressed in 4.3. This section addresses the limitations of the DSL for high turbidity conditions and reviews the field conditions for which the DSL may be beneficial.

Chapter 5 applies these observations, and proposes a series of recommendations regarding the application and design of the DSL, the design and operation of the BioSand water Filter (BSF), and topics for further study.

All of the raw data from daily filter operations are presented in tables in the appropriate appendices.

4.1 Feed Water Results & Observations

One of the major tasks in this project was to duplicate the highly stable high turbidity water sources observed in field conditions. This was important in order to ensure that the laboratory results would be applicable to field conditions. The methodology of determining a procedure for manufacturing feed water is described in Section 3.1, along with the procedure itself. This procedure was used throughout the ripening phase and all three experimental phases.

In the process of developing a feed water manufacture procedure, several potentially useful observations were made. Several experiments were conducted using the feed water to further clarify these observations. Specifically, it was found that slightly basic pH and low ionic concentrations both contribute to particle stability. These observations are elaborated in the following sections.

4.1.1 Particle Stability at High pH

Based on the fabric map of kaolinite (Section 2.4), it was also expected that the stability of kaolinite particles at a constant ionic concentration would vary with pH. Specifically, at low ionic concentration it was expected that particle stability would be greatest above a pH of 7.2, and would drop as pH was lowered. This behavior was observed as expected, and is demonstrated from two sets of data. The first set is taken from the daily operation of the filters, isolating the particle stability and comparing that against pH. The second set is taken from an experiment conducted during the process of developing a feed water manufacture procedure.

The feed water was manufactured as consistently as possible. However, due to seasonal changes in water quality, the required dose of sodium hydroxide to obtain the same response varied from

day to day (see Table 28, Table 30, and Table 32). The target pH for the feed water was just above 7.2. Adjusting the pH was the first step in manufacturing feed water, and the pH invariably drifted throughout the manufacture process (for discussion, see Section 3.1.3). The final pH was recorded, along with the final turbidity.

Before making observations based on the average turbidity reduction across the BSF's, it should be noted that the average turbidity reduction appears to be the most reliable parameter to isolate the particle stability from day to day. The turbidity reduction from any one filter depended on many different variables, including the particle stability, how recently it was maintained, and the flow rate. The average turbidity reduction across all five filters was used to isolate the particle stability. Average turbidity reduction each day was calculated for all filters except those maintained the same day. Also, filter 5a was removed from the calculations until the day it was first maintained, due to unusually high turbidity reduction rates (see discussion in Section 4.2.1).

In Figure 5, the final pH of the feed water (prior to pouring through the filters) is plotted against the average turbidity reduction.

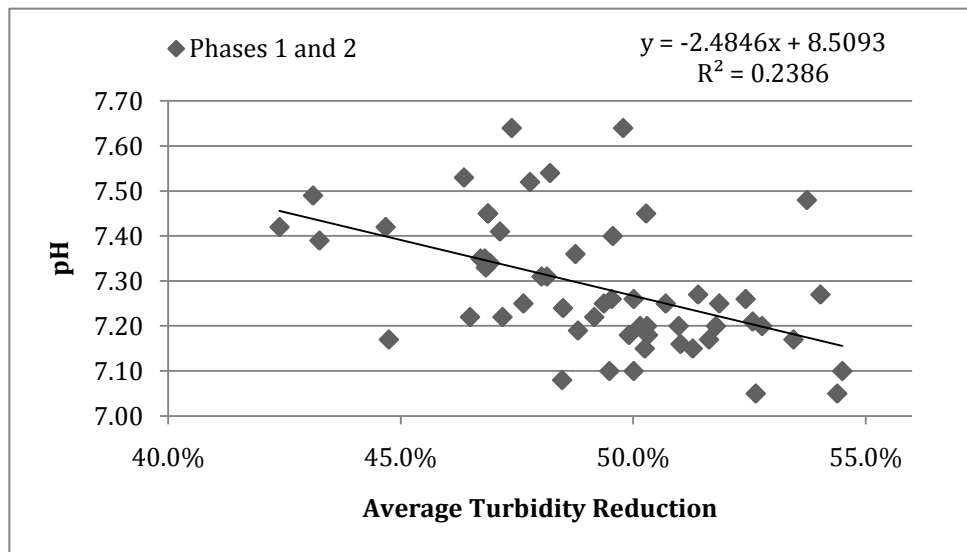


Figure 5 – Feed Water pH and Filter Average Turbidity Reduction

There appears to be a noticeable trend, suggesting that lower pH is associated with higher treatment efficiency, even within this small range.

During the process of developing a procedure for manufacturing feed water, an experiment was conducted to determine the effect of a wide range of pH on particle stability. This experiment was conducted by adjusting the pH of 1 L of water from a nearby lake (Indian Lake), and mixing with 0.250 g of kaolinite clay in several beakers. The beakers were allowed to settle for a period of 120 h, and the turbidity before and after were measured. The results are presented in Figure 6.

It is interesting that the initial turbidities had almost as wide of a spread as the final turbidities. This is a reminder of the fact that turbidity depends highly on the particle dispersion and interaction, not just on the total mass of suspended particles. In spite of this complication, the percent reduction of turbidity increases significantly at lower pH values of 5 and 4.

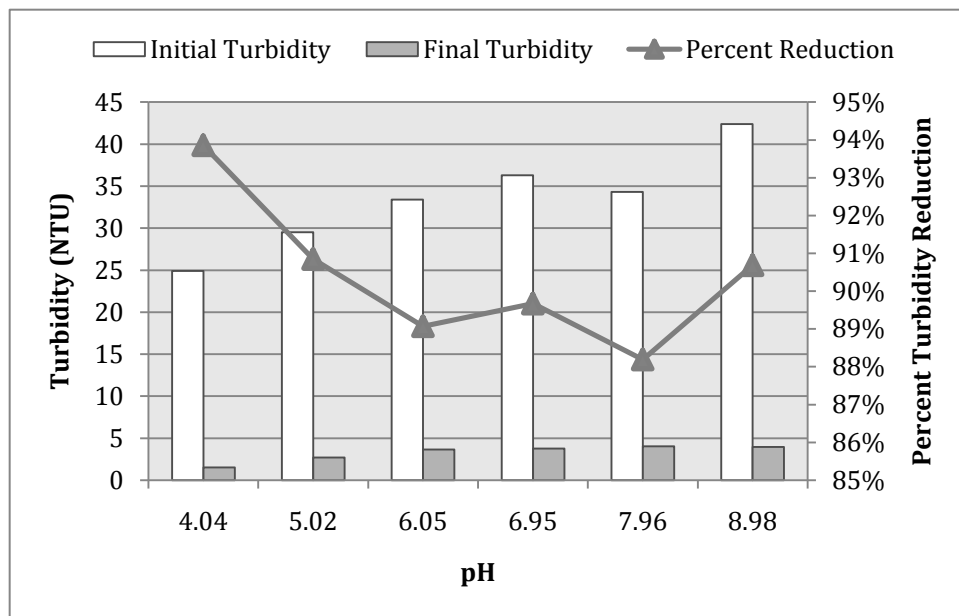


Figure 6 – pH and Particle Stability

This experiment was later repeated using the same feed water that was manufactured for the filters, and a pH range of 5.5 to 7.5. In this case, no change in settling turbidity reduction was observed over a period of one week. However, after the test was concluded, the pH of the acidic beakers were checked, and observed to have drifted back up above 6.5. It is unknown why this drift occurred. It may be due to adjustments in the equilibrium reactions of the carbonate group. Some indications hint that reactions involving the kaolinite may provide some buffering

capacity. Specifically, in phase 3 when the target turbidity was increased to 1000 NTU, using the same water and the pH adjustment technique consistently resulted in a significantly lower final pH.

In summary, particle destabilization with lower pH was observed from both sets of data and is consistent with the theory of the fabric map of kaolinite, described in Section 2.4. Observing the high variability of the data, it is clear that many other factors are important. Nevertheless, this trend supports the theory that lower pH decreases the stability of suspended kaolinite clay particles in water. This observation appears to be potentially useful in the treatment of high turbidity drinking water supplies.

4.1.2 Particle Stability at Low Ionic Concentration

Based on the fabric map of kaolinite (Section 2.4), it was also expected that the stability of kaolinite particles would vary with ionic concentration. Specifically, it was expected that given a pH above 7.2, particles would be less stable at high ionic concentrations. This behavior was observed, both in the development of a feed water manufacturing procedure, and in several experiments conducted with the manufactured feed water.

When first attempting to develop a procedure for manufacturing the feed water, it was not known that ionic concentration might be important. Several types of particulate additives were used, including sand, loam, montmorillonite clay, and kaolinite clay. Tests were initially conducted with laboratory-grade water, and water from two local surface water bodies, Salisbury Pond and Indian Lake. With all side-by-side tests, all suspended particles were observed to be most stable in laboratory-grade water, and least stable in Salisbury Pond water. One test even used the blender technique described in Section 3.1.3. None of these initial experiments resulted in the desired level of stability, even for Indian Lake water with kaolinite when pH was adjusted as low as 4 and as high as 9.

When the literature review revealed the article by Palomino and Santamarina (2005) on the fabric map of kaolinite, the prior experiments were reexamined. Salisbury Pond water was found

to have approximately 250 mg/L of NaCl, and Indian Lake, 160 mg/L NaCl. Additionally, Na⁺ and Cl⁻ were found to be the dominant ions contributing to ionic concentration.

Based on this conclusion, further testing used water from Wachusett Reservoir, which is known to be a highly pristine water supply. Testing revealed approximately 45 mg/L NaCl. This water was found to result in significantly higher particle stability than both Salisbury Pond water and Indian Lake. Combining the low ionic concentration of Wachusett Reservoir with pH adjust and blending was found to produce the desired particle stability, very similar to the feed water criteria described in Table 1.

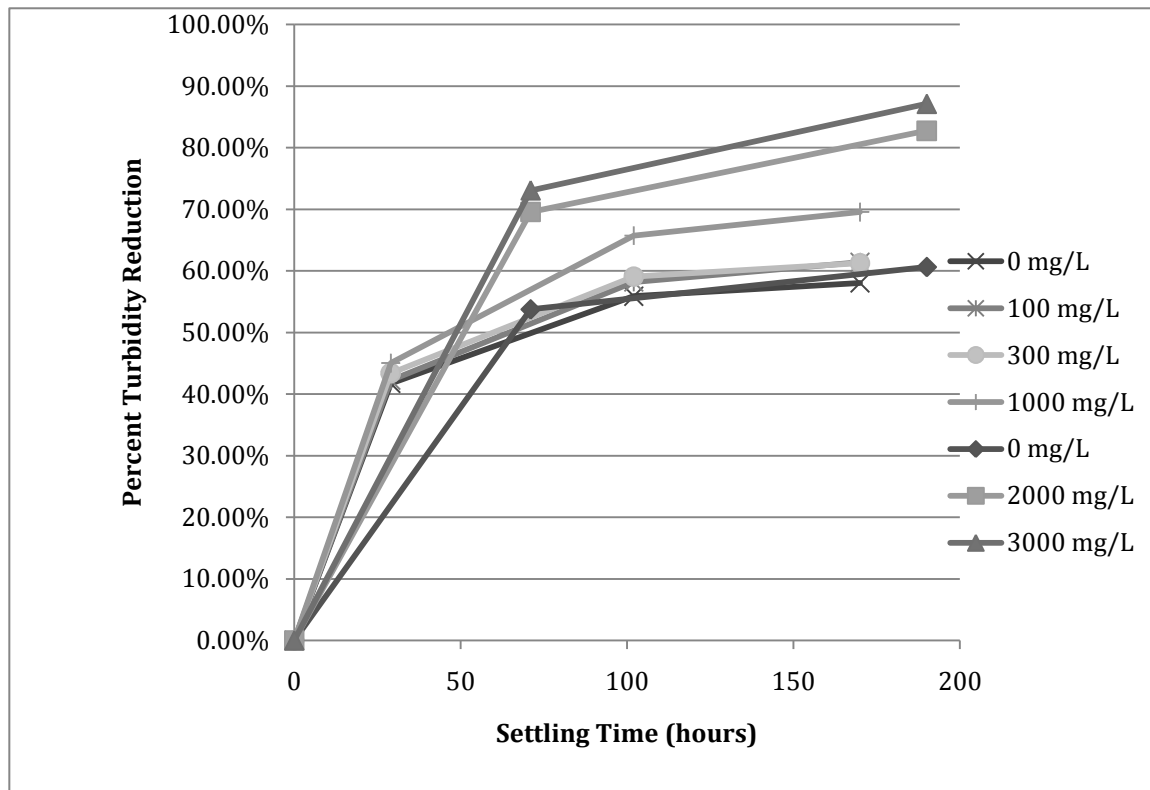


Figure 7 – NaCl addition to Feed Water

Further experiments examined the impact of adding NaCl to the manufactured feed water, after pH adjustment and kaolinite addition. Figure 7 presents the results of these experiments. The results confirm that ionic concentration has a significant influence on the stability of kaolinite clay particles in water.

It is interesting to observe that 100 mg/L and 300 mg/L had relatively small impacts compared to the impacts observed while manufacturing the feed water. This may indicate that the ionic concentration when kaolinite is initially dispersed in the water is more important than changes in ionic concentration after the kaolinite is effectively dispersed. However, more carefully controlled experiments are necessary in order to confirm this observation.

The effects of low ionic concentration and high pH on kaolinite were used to duplicate the highly stable high turbidity conditions reported from the field. Successfully duplicating these conditions provides the basis for the filter operation experiments described in the following section.

4.2 Filter Operation Results

This section presents the results from daily operations of the five laboratory filters. This section is divided into four subsections, with the first three presenting the results from each of the three phases. The three phases were associated with differences in DSL configuration and operational parameters strategically selected for each filter. See Table 3 through Table 5 on page 35 for details on the filter configurations.

The fourth subsection presents the results of particular measurements that were made not according to the daily schedule. The filters were operated normally on these days, but additional samples were taken to obtain the presented data. These measurements include turbidity profiles, a DO profile, cations, anions, filtrate pH, alkalinity, total solids, suspended solids, dissolved solids, and particle counts.

Many observations can be made based on these results. Section 4.3 draws out specific observations related to the DSL design and filter operation, utilizing all of the following data. Sharp peaks in both flow rate and turbidity are generally associated with filter maintenance. Filter maintenance dates are documented in Table 7.

4.2.1 Phase 1 Daily Measurements

This section presents the data collected during daily operation of the filters in phase 1, which began on January 4th and concluded on February 23rd. For details on the configurations used in

phase 1, refer to Table 3 on page 35. Phase 1 focused on low ranges of DSL sand thickness, and filter 4a was used to determine the effects of greater daily flow volume.

Table 2 and Table 8 have been duplicated here for reference.

Table 2 – Filter Label Code

Expression	Example	Meaning
#a/b/c	1a	filter number and phase
Control	Control	control filter – no DSL modification
#cm	3cm	depth of DSL fine sand layer in cm
#+#	7+3	depth of fine sand in cm + depth of supernatant water in cm
#L	3L	daily water volume in L (default is 2 L or 2.25 L if not specified)

Table 8 – Maintenance Calendar

Date	Filter No.	Maintenance Cause
12/7/09	All	Initial Filter Setup
1/26/10	5	Flow Rate
1/27/10	4	Flow Rate
2/8/10	3	Flow Rate
2/17/10	5	Flow Rate
2/24/10	2, 3, 4	Filter Reconfiguration
3/17/10	1, 5	Flow Rate

The behavior observed by filter 5a prior to the first maintenance on 1/26 was not repeated by any filter at any point. Very high turbidity reduction was associated with rapid clogging (as expected for high suspended particle deposition). The most likely explanation seems to be that perhaps the sand in the DSL was slightly more compacted at the top than the other filters. This would explain both the higher treatment and the reset back to what was normal for all other filters after the first maintenance.

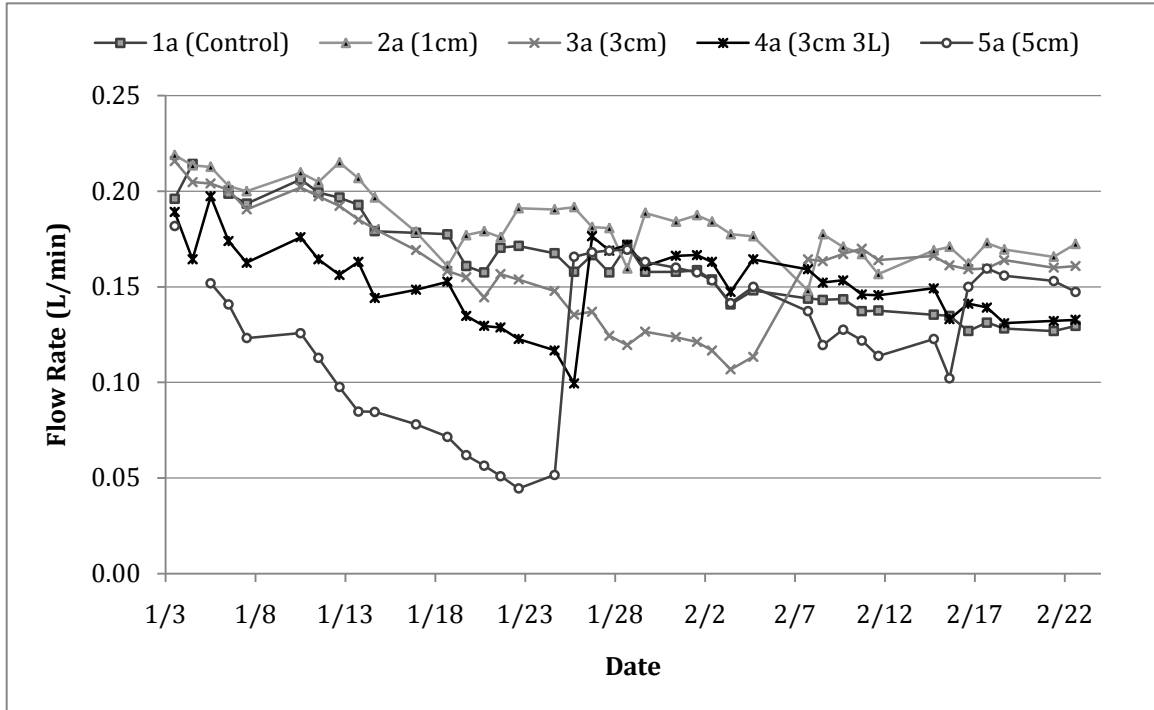


Figure 8 – Phase 1 Flow Rate

Figure 8 presents the flow rates of phase 1 filters by date. Prior to the beginning of phase 1, the flow rates start out all around 0.23 L/min. Flow rates drop most quickly for filters with the thickest DSL's. Major jumps in flow rate are associated with filter maintenance.

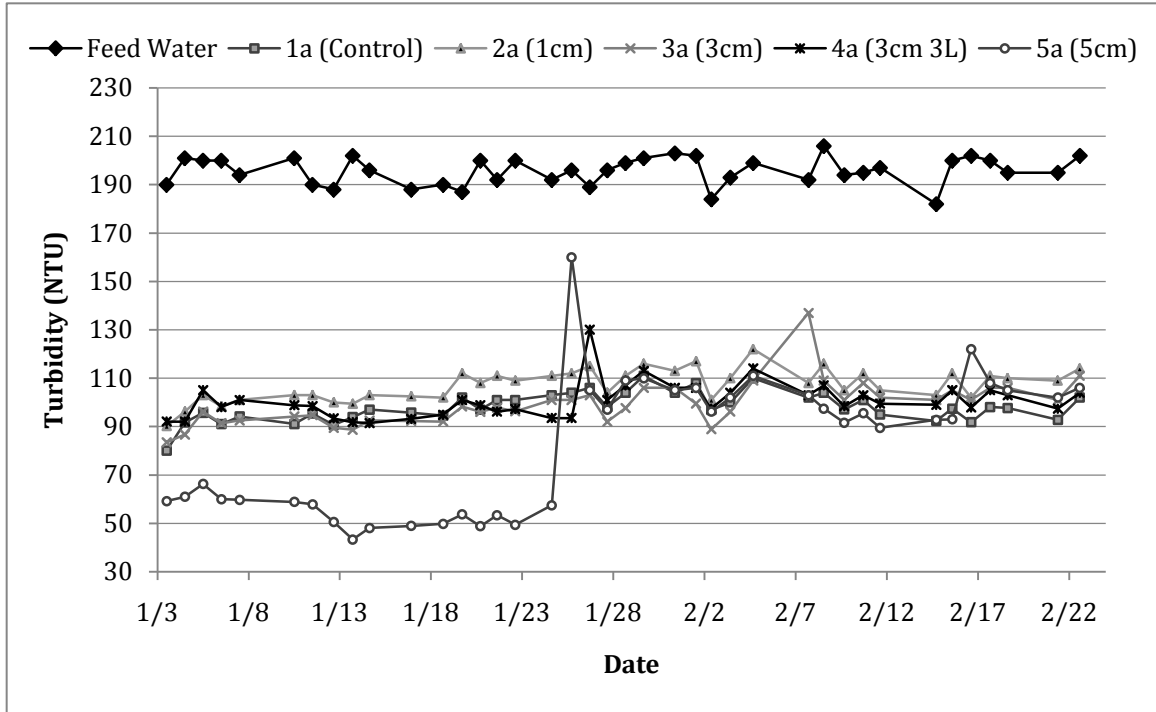


Figure 9 – Phase 1 Turbidity by Date

Figure 9 and Figure 10 present the turbidities of phase 1 filters by date, first according to direct measurements, and next according to calculated percent reductions. Major peaks or valleys are associated with filter maintenance.

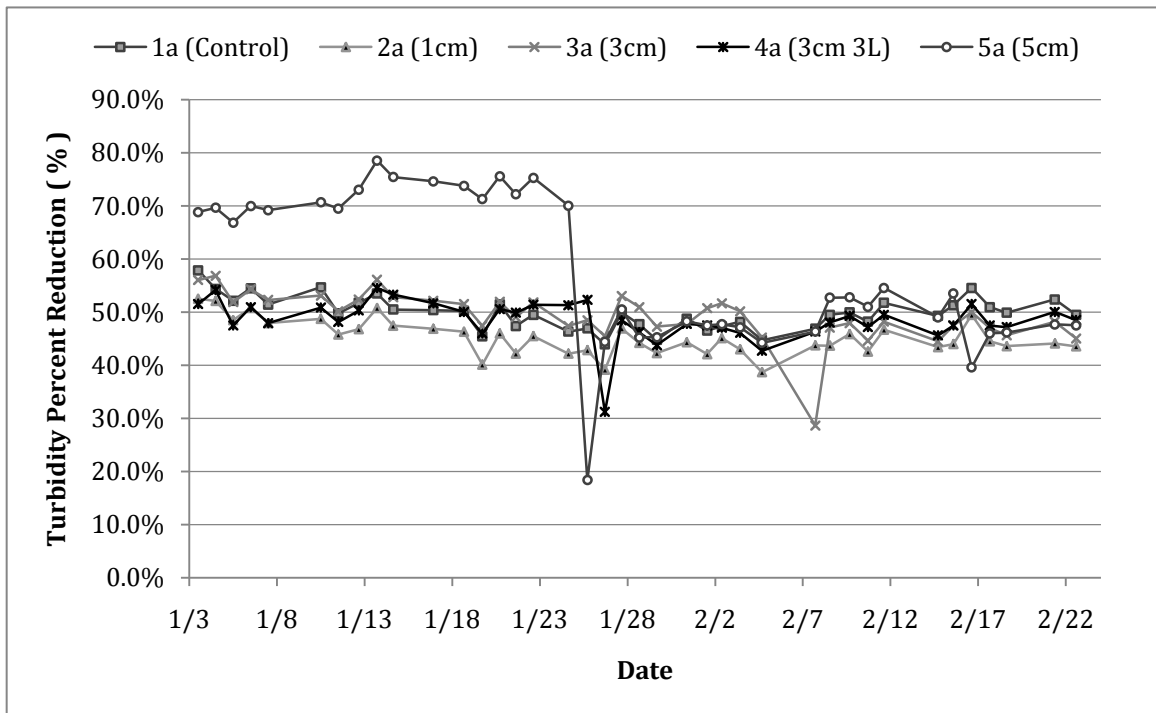


Figure 10 – Phase 1 Turbidity Percent Reduction by Date

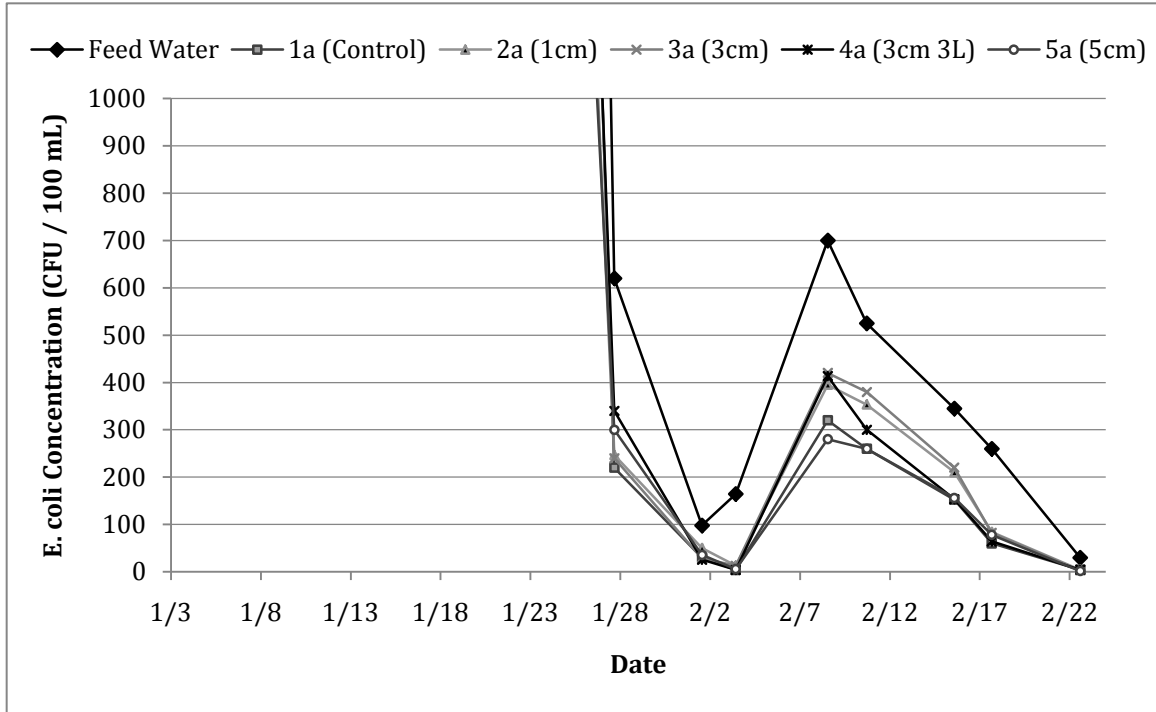


Figure 11 – Phase 1 *E. coli* by Date

Figure 11 and Figure 12 present the *E. coli* measurements of phase 1 by date, according to measured values and percent reductions. Percent reductions are much more consistent than total counts. Very low *E. coli* concentrations are statistically unreliable, and are associated with wastewater stock that was outdated, which happened more quickly with the change of seasons.

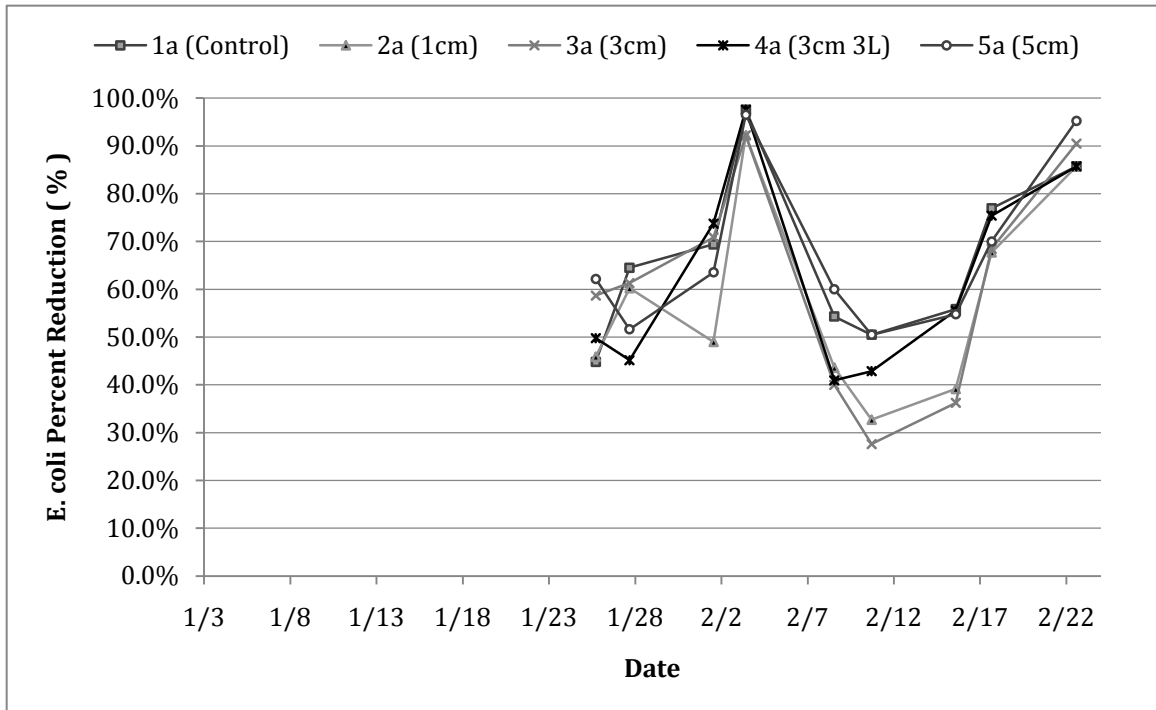


Figure 12 – Phase 1 *E. coli* Percent Reduction by Date

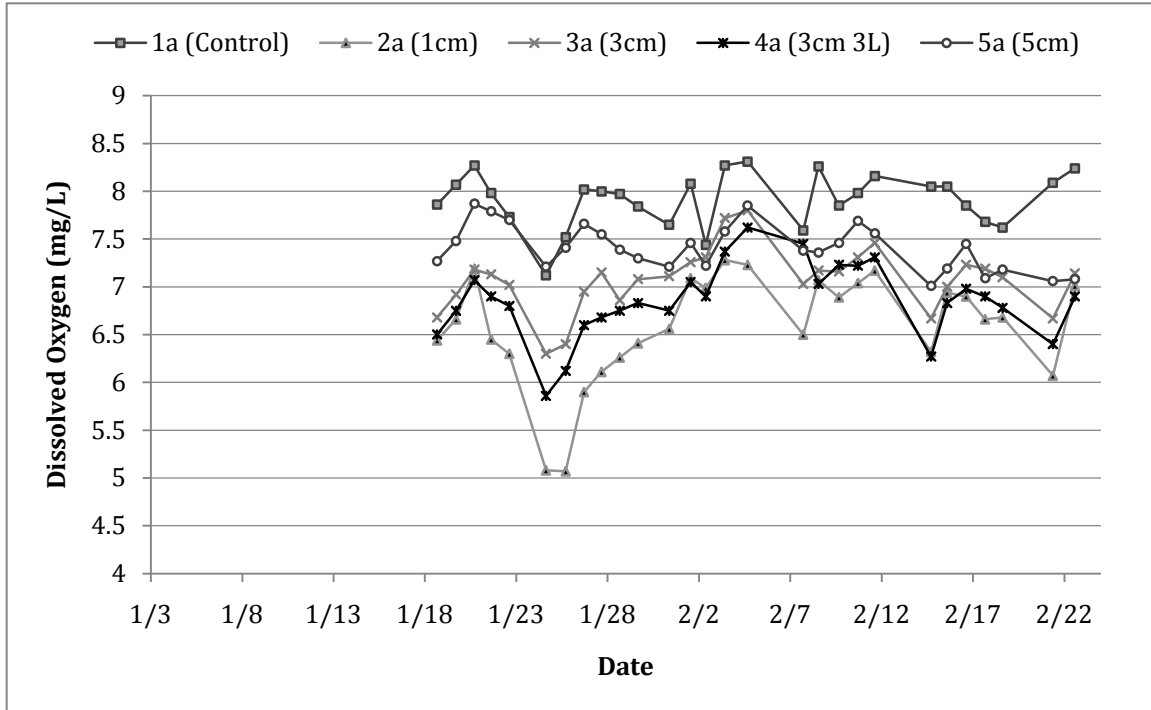


Figure 13 – Phase 1 Sample Port Dissolved Oxygen by Date

Figure 13 and Figure 14 present the DO measurements by date from the sample port and the filtrate at 0.75 L. The consistent drops each weekend in the filtrate are associated with high pause times. The lack of this pattern in the sample port suggests that the filtrate DO is more critical than the sample port DO.

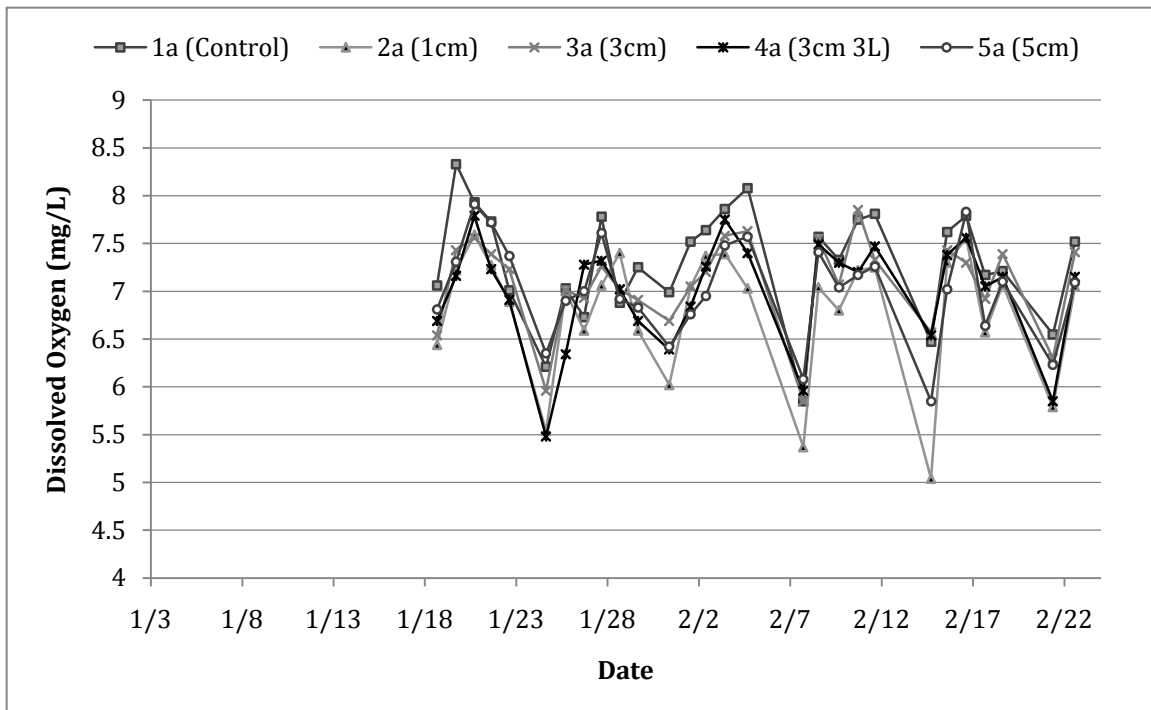


Figure 14 – Phase 1 Filter Effluent Dissolved Oxygen by Date

4.2.2 Phase 2 Daily Measurements

This section presents the data collected during daily operation of the filters in phase 2, which began on February 24th and concluded on March 22nd. For details on the configurations used in phase 2, refer to Table 4 on page 35. Phase 2 focused on higher ranges of DSL sand thickness with filter 3b using a thicker supernatant water layer as well.

Table 2 and Table 8 have been duplicated here for reference.

Table 2 – Filter Label Code

Expression	Example	Meaning
#a/b/c	1a	filter number and phase
Control	Control	control filter – no DSL modification
#cm	3cm	depth of DSL fine sand layer in cm
#+#	7+3	depth of fine sand in cm + depth of supernatant water in cm
#L	3L	daily water volume in L (default is 2 L or 2.25 L if not specified)

Table 8 – Maintenance Calendar

Date	Filter No.	Maintenance Cause
12/7/09	All	Initial Filter Setup
1/26/10	5	Flow Rate
1/27/10	4	Flow Rate
2/8/10	3	Flow Rate
2/17/10	5	Flow Rate
2/24/10	2, 3, 4	Filter Reconfiguration
3/17/10	1, 5	Flow Rate

Filters 2, 3, and 4 were maintained between phases 1 and 2 not due to low flow rate but rather in the process of reconfiguring the filters for the new phase. Maintenance of the lower sand layers were conducted along with reconstruction of the DSL's with new sand. The lower flow rates after maintenance reflect changes in the hydraulic head loss (more sand) and the available head (less freeboard). The next several days after maintenance, unusually low turbidity and DO as well as high *E. coli* reduction were observed for all three maintained filters. This behavior is

unexplained, and appears to be associated with the new DSL. After several days, all parameters returned to typical values.

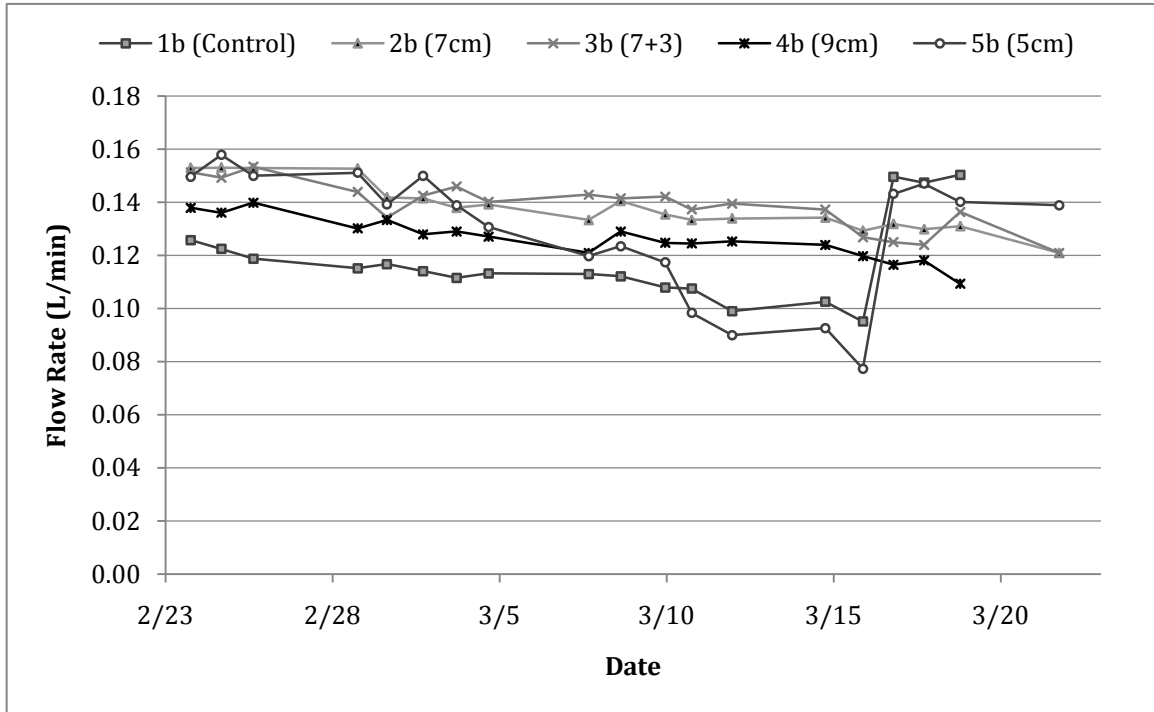


Figure 15 – Phase 2 Flow Rate

Figure 15 presents the flow rates for phase 2. Note that the flow rates are all below 0.16 L/min, even after filters 1b and 5b were maintained on 3/17.

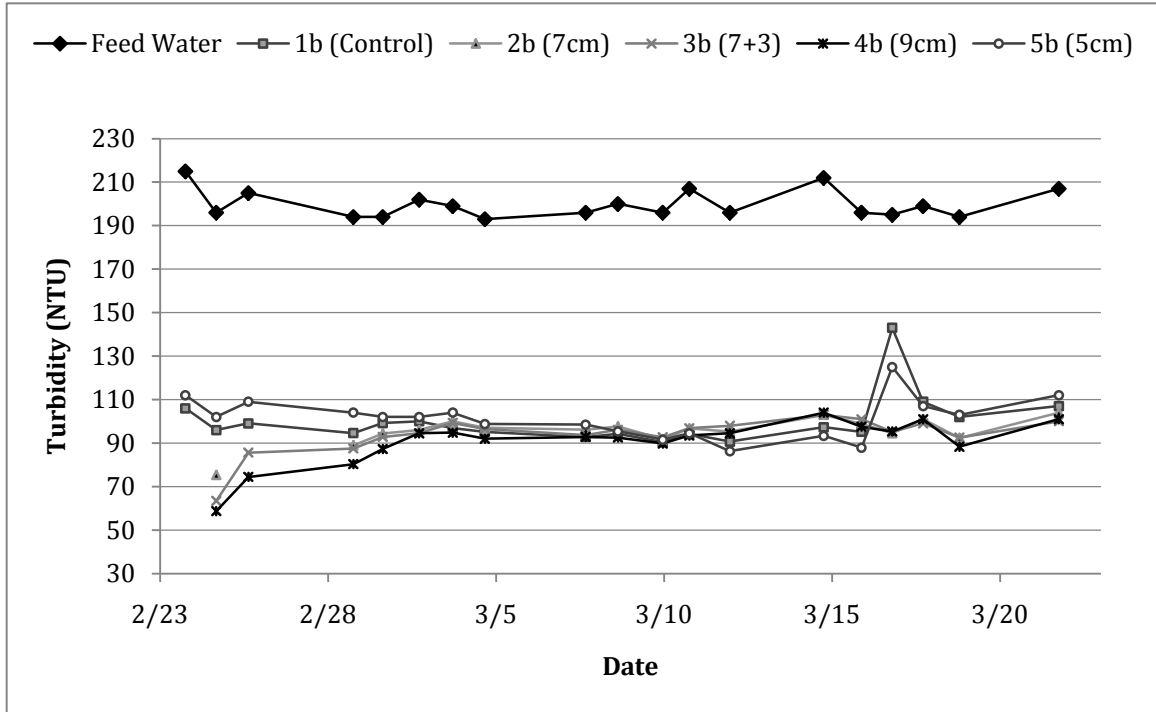


Figure 16 – Phase 2 Turbidity by Date

Figure 16 and Figure 17 present the turbidities measured during phase 2, according to measured values and percent reductions. The strange behavior after reconfiguration of filters 2, 3, and 4 stand out. Notice that the turbidity peak for maintenance of filter 1b was higher than filter 5b. This is primary observed benefit of the Dual Sand Layer modification to the BioSand Filter.

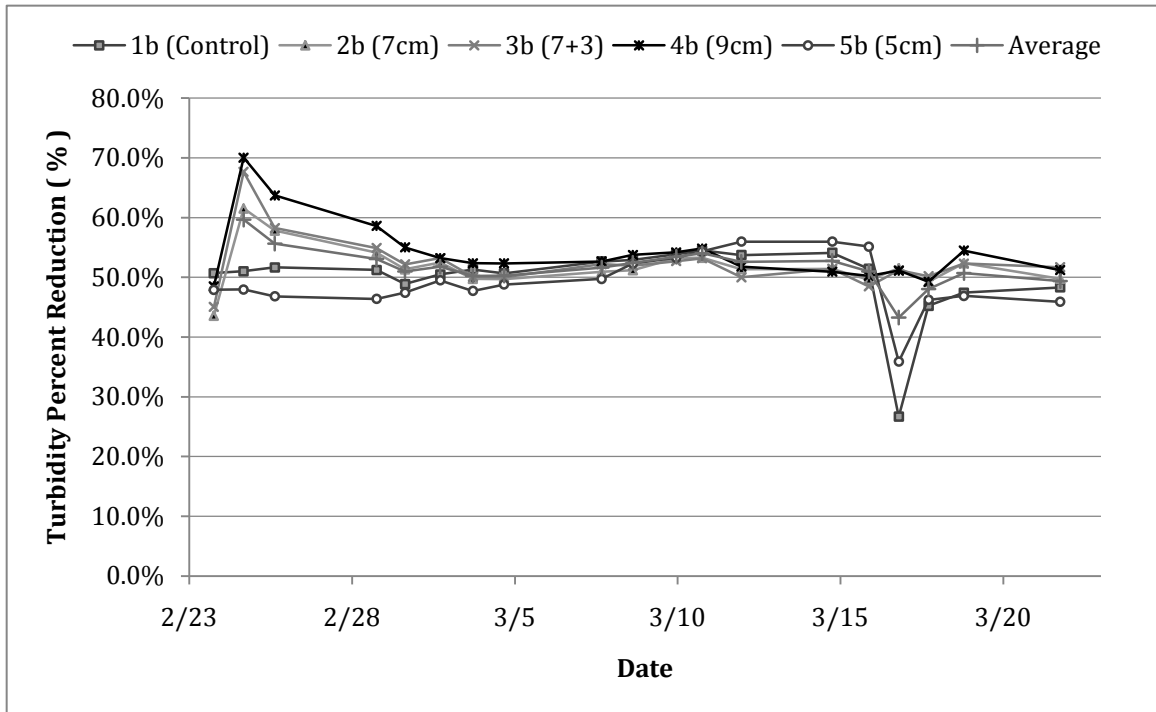


Figure 17 – Phase 2 Turbidity Percent Reduction by Date

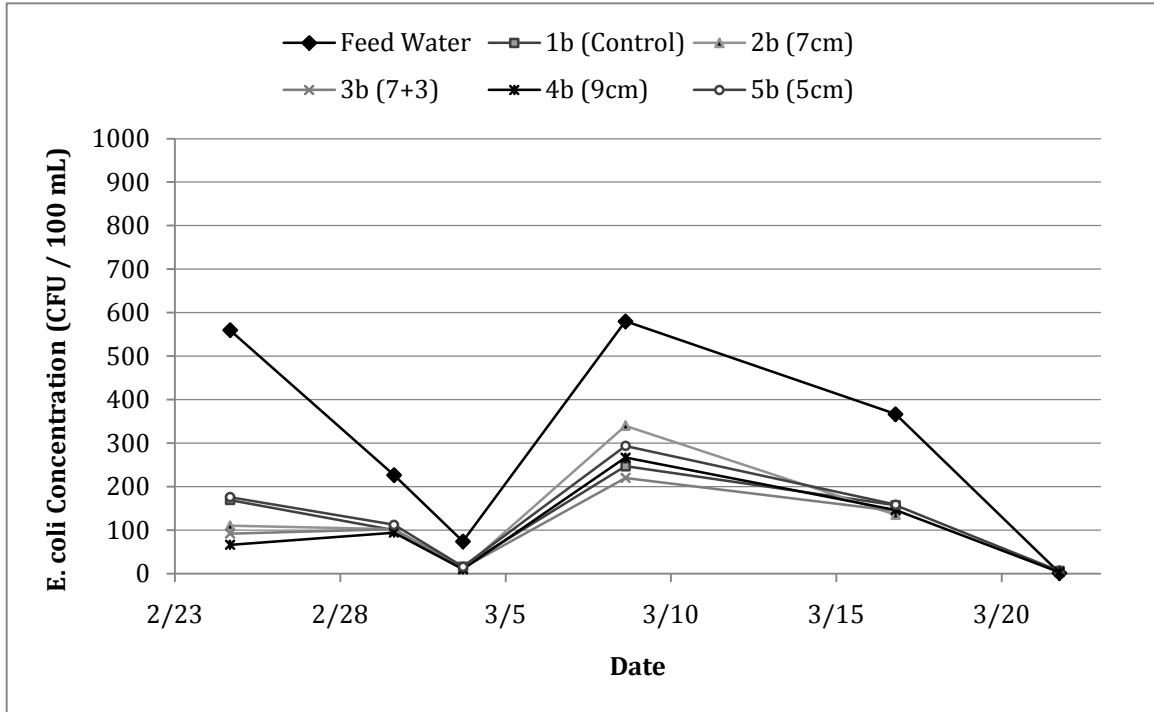


Figure 18 – Phase 2 *E. coli* by Date

Figure 18 and Figure 19 present the *E. coli* concentrations for phase 2 according to measurement and percent reduction. The strange behavior of filters 2, 3, and 4 after reconfiguration is again observed. Note the relative closeness of all five filters each day compared to the changes between one measurement and the next.

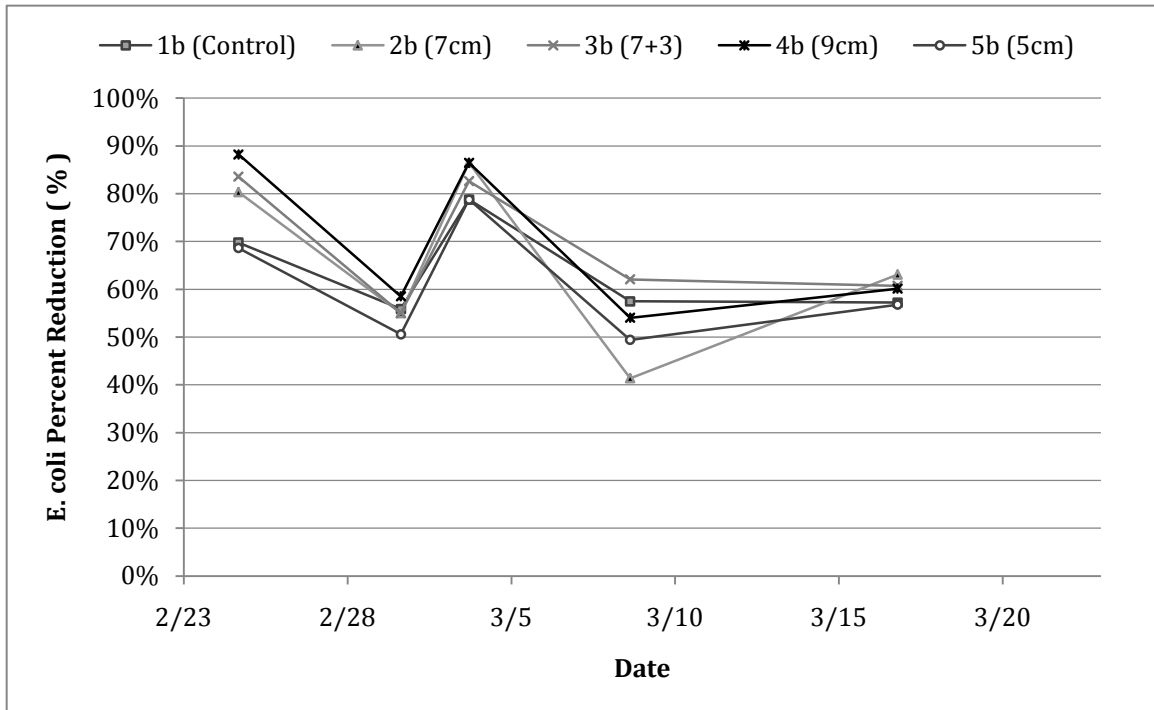


Figure 19 – Phase 2 *E. coli* Percent Reduction by Date

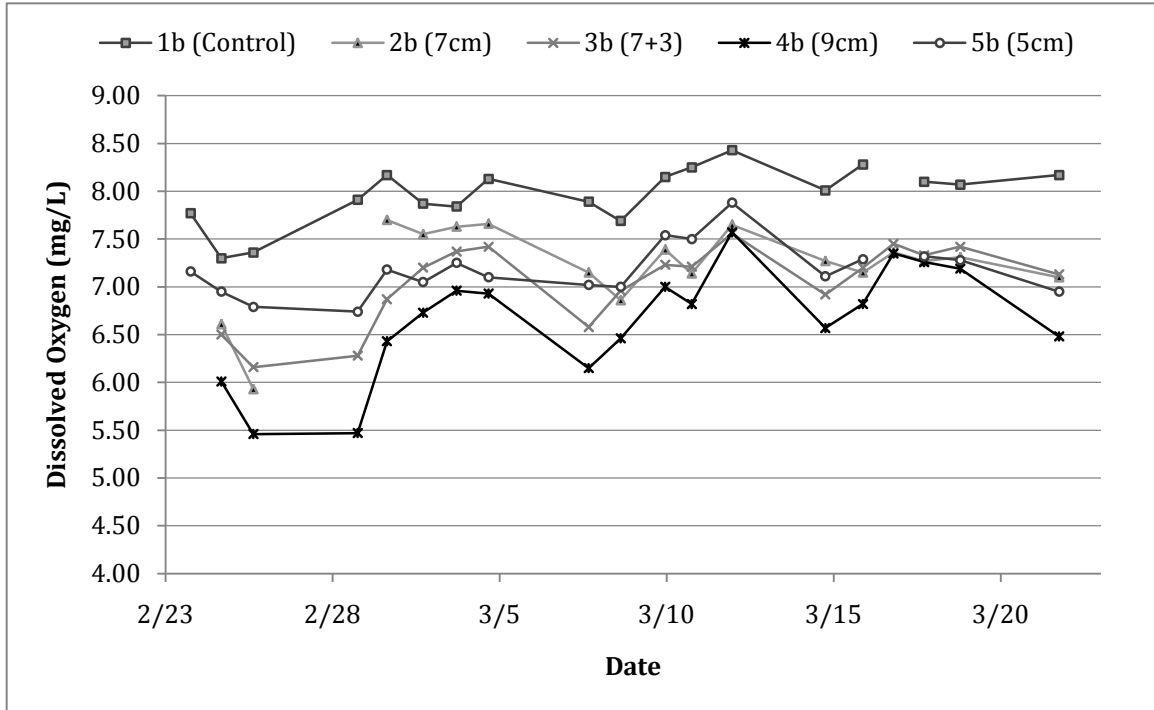


Figure 20 – Phase 2 Sample Port Dissolved Oxygen by Date

Figure 20 and Figure 21 present the DO in both the sample port and filter effluent. Definite patterns are observed between the DO each day and the total thickness of the DSL sand and supernatant water thickness. Again, note the strange behavior the first three days of operation for filters 2, 3, and 4 after reconfiguration.

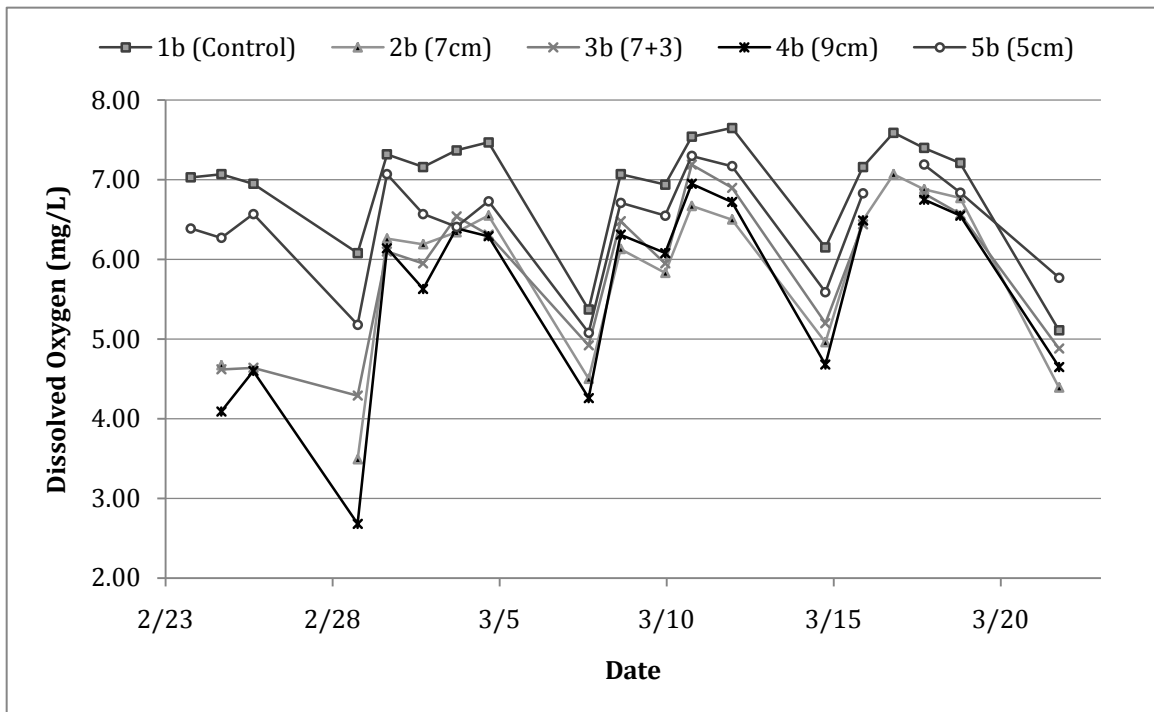


Figure 21 – Phase 2 Filter Effluent Dissolved Oxygen by Date

4.2.3 Phase 3 Daily Measurements

This section presents the data collected during daily operation of the filters in phase 3, which began on March 23rd and concluded on April 7th. For details on the configurations used in phase 3, refer to Table 5 on page 35. Phase 3 was used to test and fine tune several of the recommendations presented in chapter 5. The daily volume of most filters was dropped to 1 L, and the supernatant of some was raised to 5 cm. This changed the flow rate measurements, and filters were not maintained during phase 3. On 3/24 and 3/25, the filter was operated with pause times of 4h, 8h, and 12h. Also, two tests were done with pause times of 72h and 95h. These provide valuable information regarding DO and turbidities at short and long pause times. Additionally, starting on 3/27, the target turbidity of the feed water was increased to 1000 NTU. Also, although a few *E. coli* measurements were taken during phase 3, most of the results were not reliable due to several complications. These data are included in Table 46 but are not presented here.

Table 2 has been duplicated here for reference.

Table 2 – Filter Label Code

Expression	Example	Meaning
#a/b/c	1a	filter number and phase
Control	Control	control filter – no DSL modification
#cm	3cm	depth of DSL fine sand layer in cm
#+#	7+3	depth of fine sand in cm + depth of supernatant water in cm
#L	3L	daily water volume in L (default is 2 L or 2.25 L if not specified)

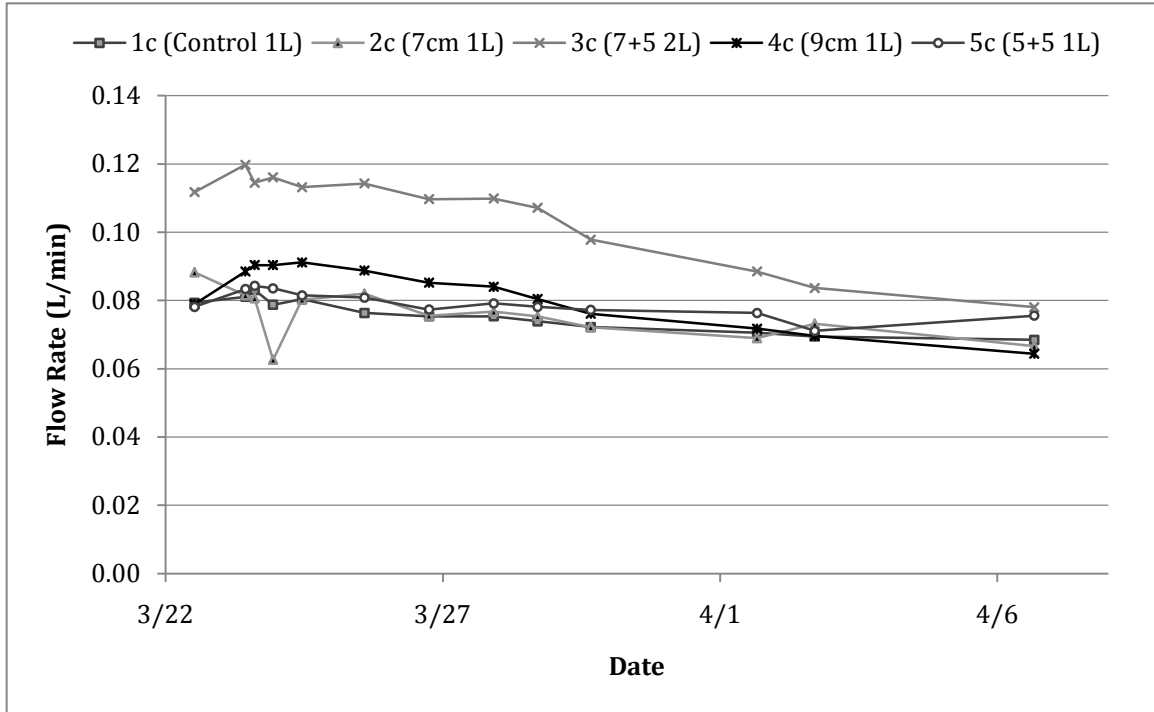


Figure 22 – Phase 3 Flow Rate

Figure 22 presents the flow rates for phase 3. Note that filter 3c was had a higher daily volume, so the average water height during operation was higher. Nevertheless, a higher clogging rate for the higher operation volume filter is observed.

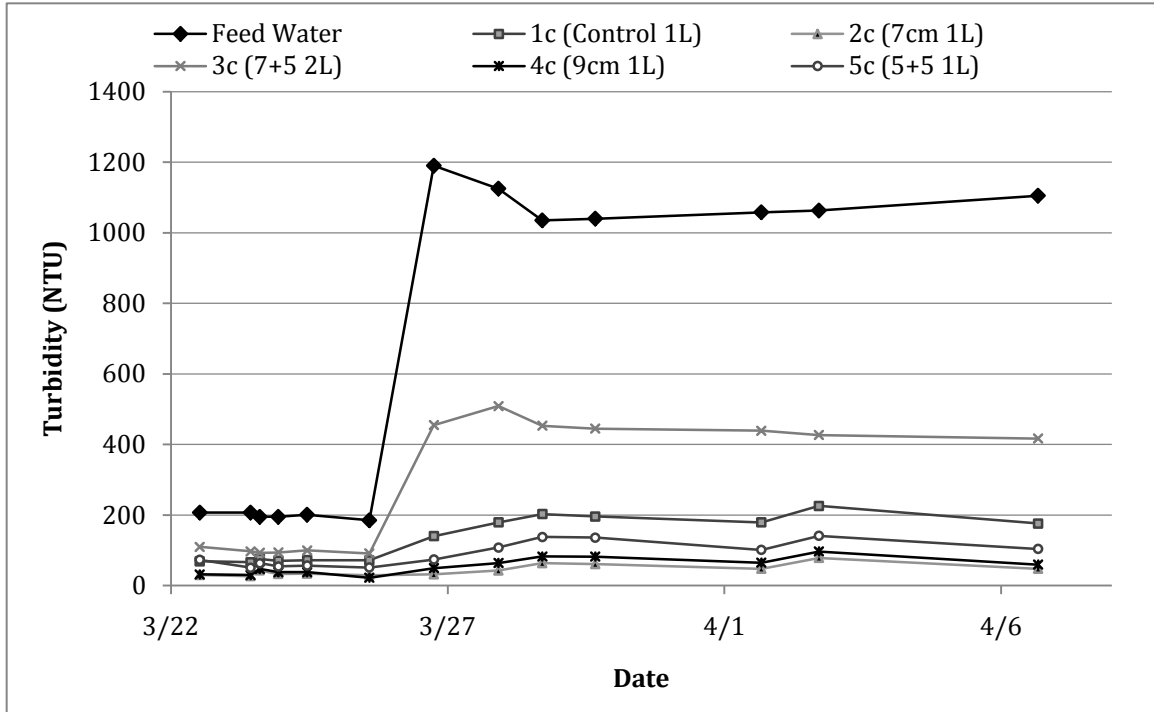


Figure 23 – Phase 3 Turbidity by Date

Figure 27 and Figure 28 present the turbidities for phase 3, both according to measured values and percent reduction. Higher percent reductions after the turbidity increased are interesting. The peak the first two days indicate flushing and mixing. Also note the 4 hour, 8 hour, and 12 hour pause time tests on 3/24 and 3/25.

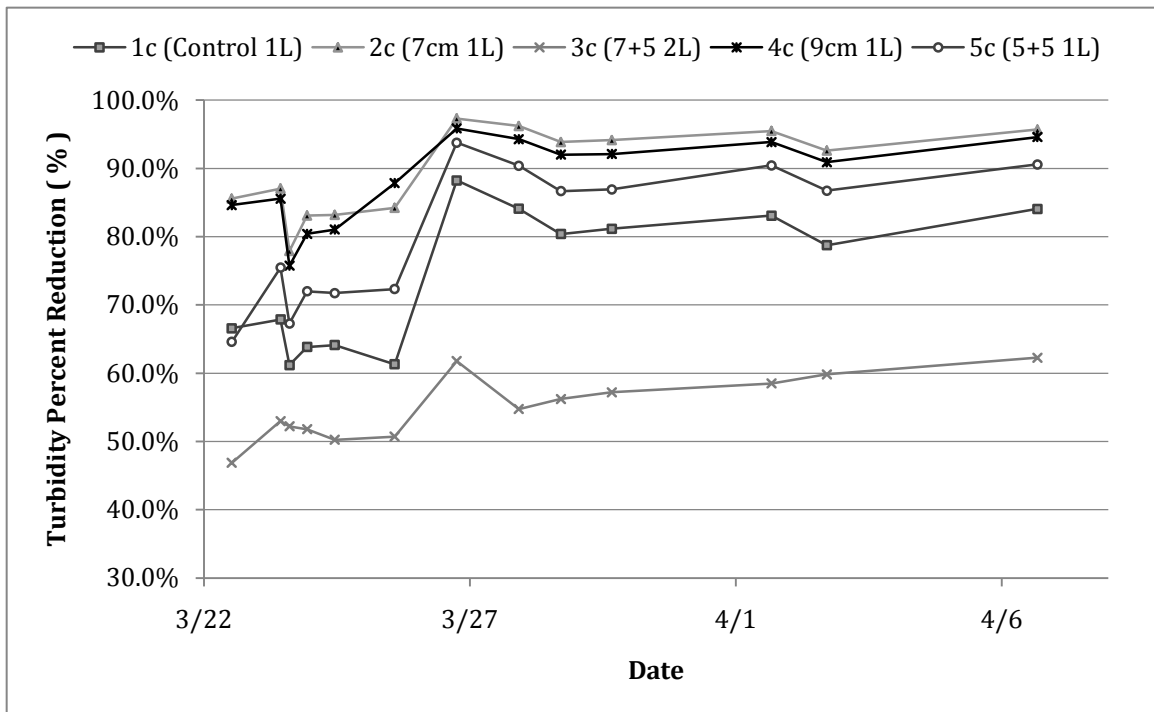


Figure 24 – Phase 3 Turbidity Percent Reduction by Date

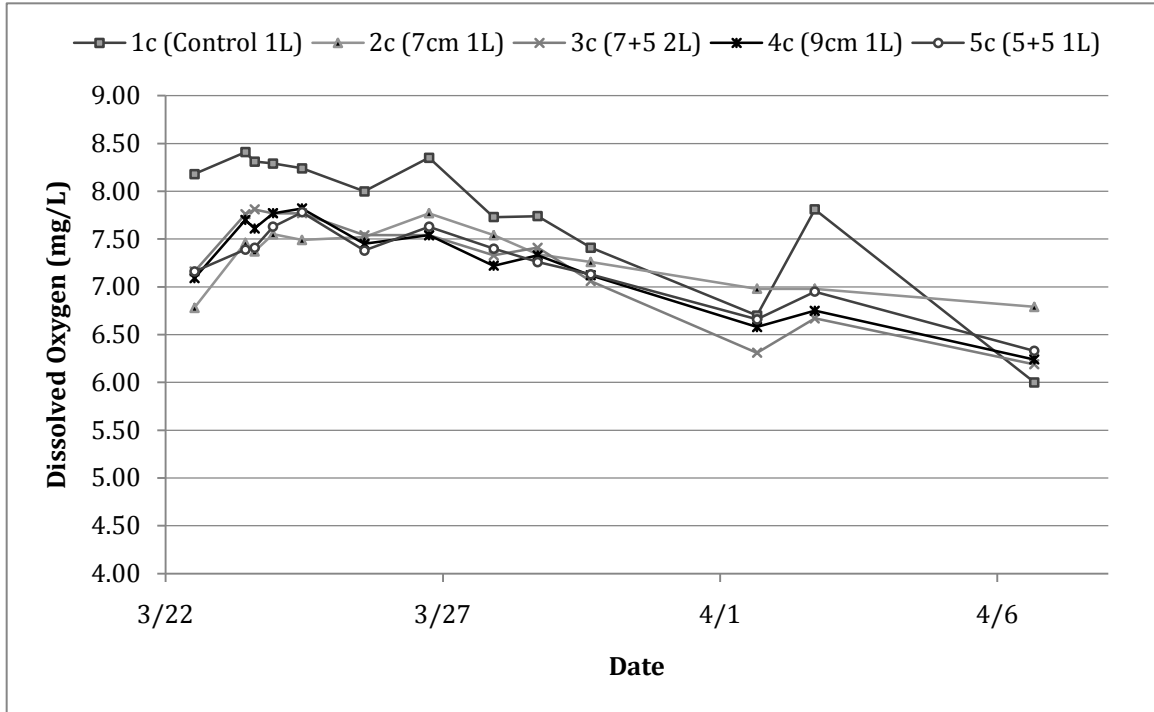


Figure 25 – Phase 3 Sample Port Dissolved Oxygen by Date

Figure 29 and Figure 30 present the dissolved oxygen concentrations for phase 3 for both the sample port and the filtrate. Note particularly the high values with the very short pause times, and the low values with the very long pause times.

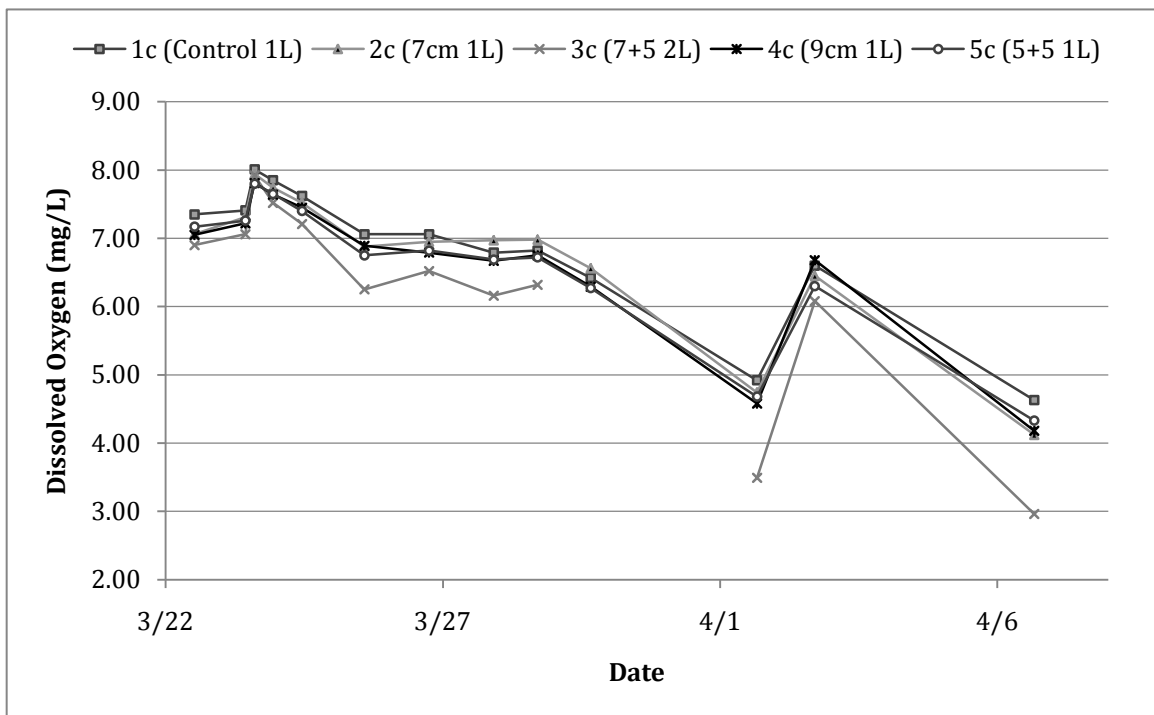


Figure 26 – Phase 3 Filter Effluent Dissolved Oxygen by Date

4.2.4 Dissolved Oxygen Correlation with Pause Time

This section presents the same DO data as in the previous, except plotted according to pause time instead of by date. Figure 27 and Figure 28 present the DO values for phase 1. A general correlation with pause time is noticed for the filtrate.

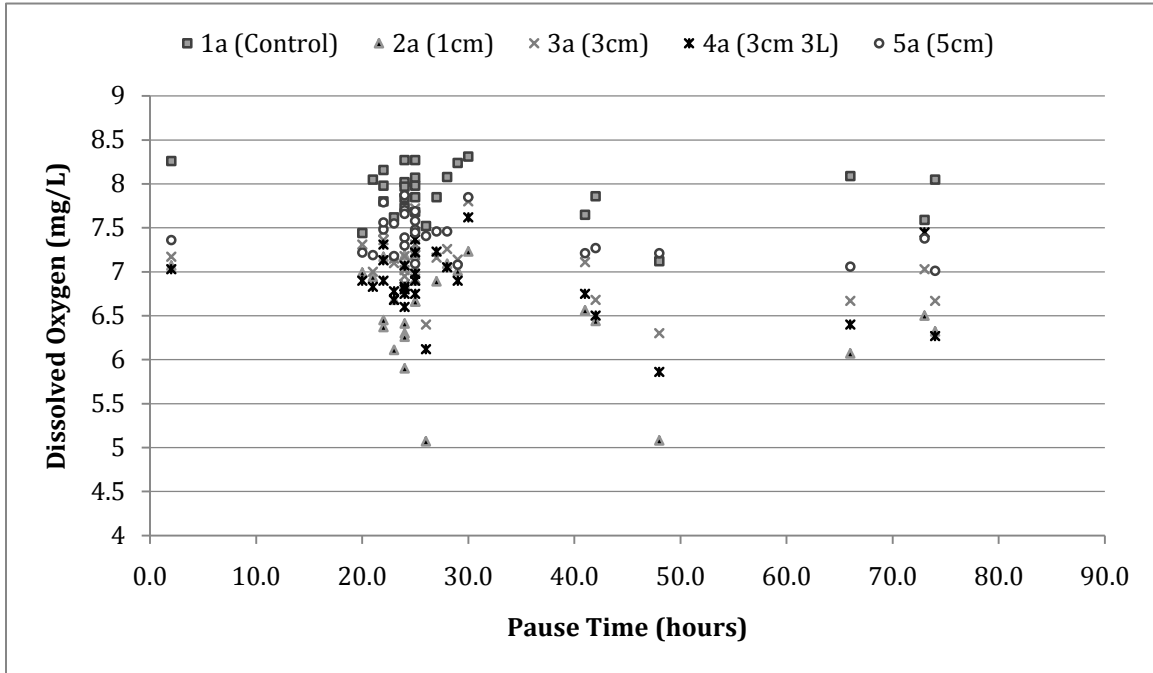


Figure 27 – Phase 1 Sample Port Dissolved Oxygen by Pause Time

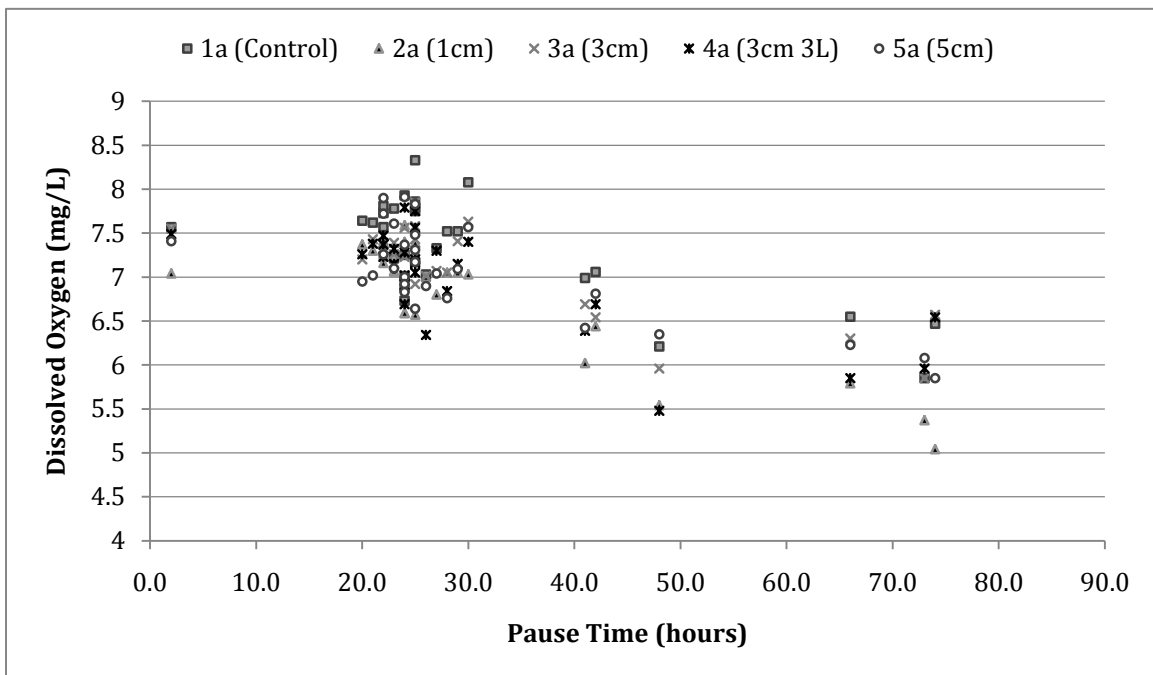


Figure 28 – Phase 1 Filter Effluent Dissolved Oxygen by Pause Time

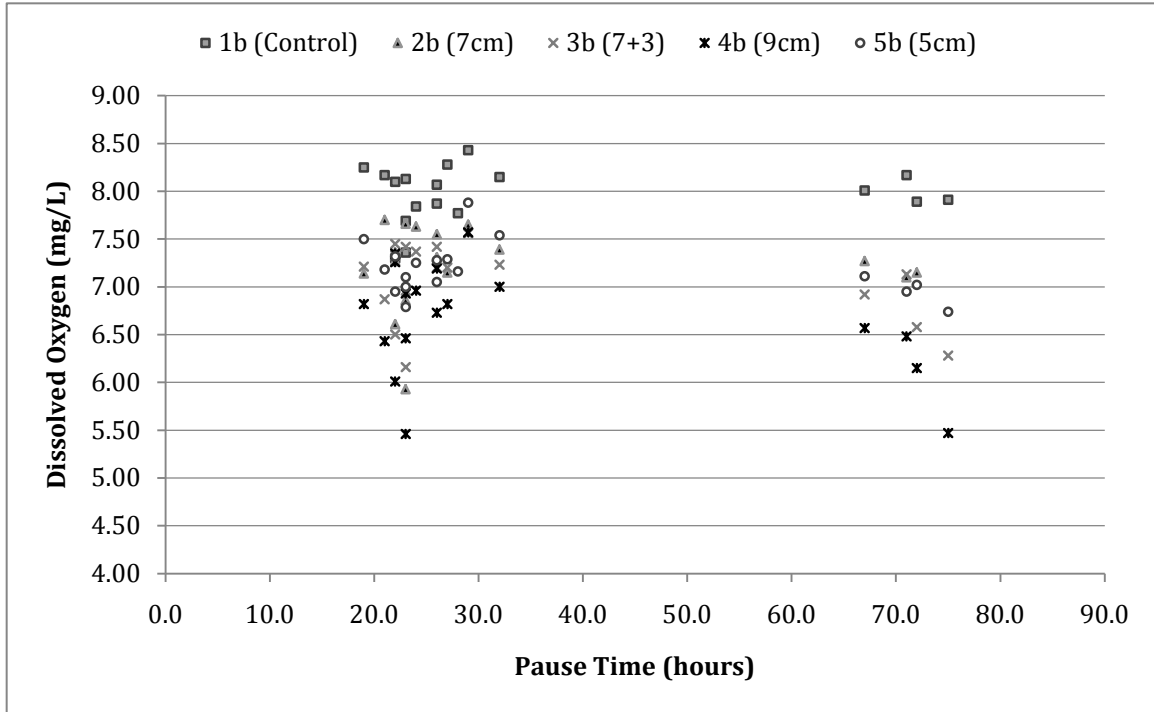


Figure 29 – Phase 2 Sample Port Dissolved Oxygen by Pause Time

Figure 29 and Figure 30 present the DO by pause time for phase 2. Here, an association is observable between DSL sand and water layer total depth and the DO, especially at high pause times but also at 24 hour pause times.

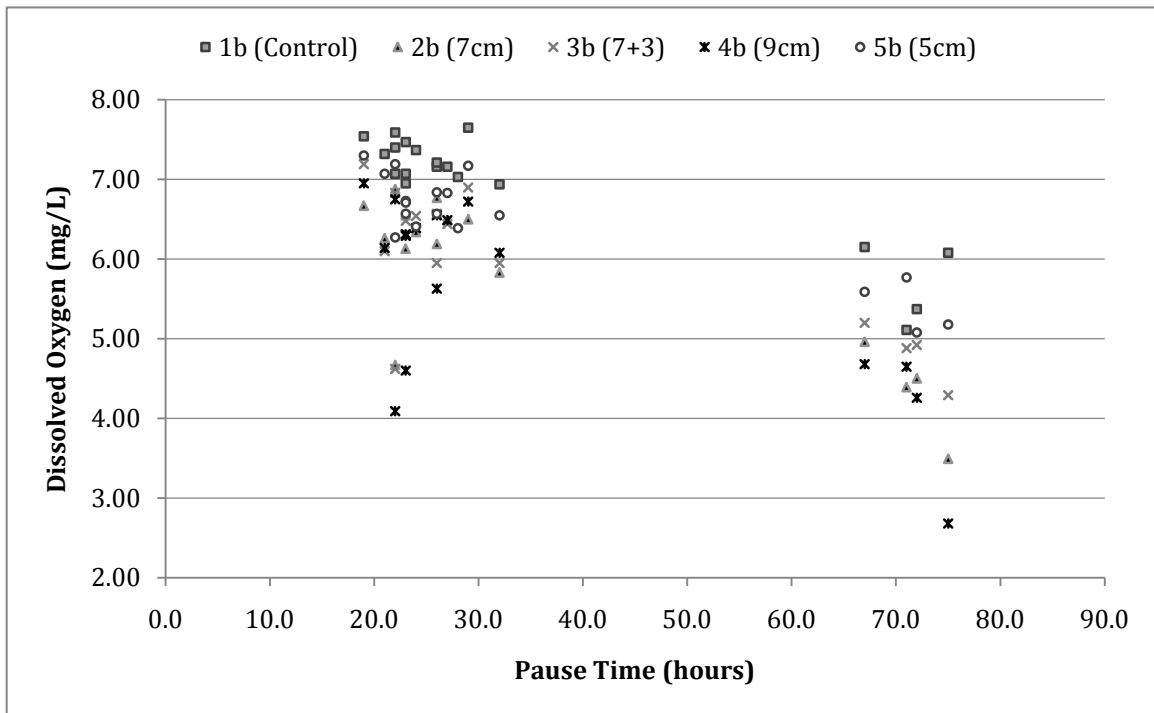


Figure 30 – Phase 2 Filter Effluent Dissolved Oxygen by Pause Time

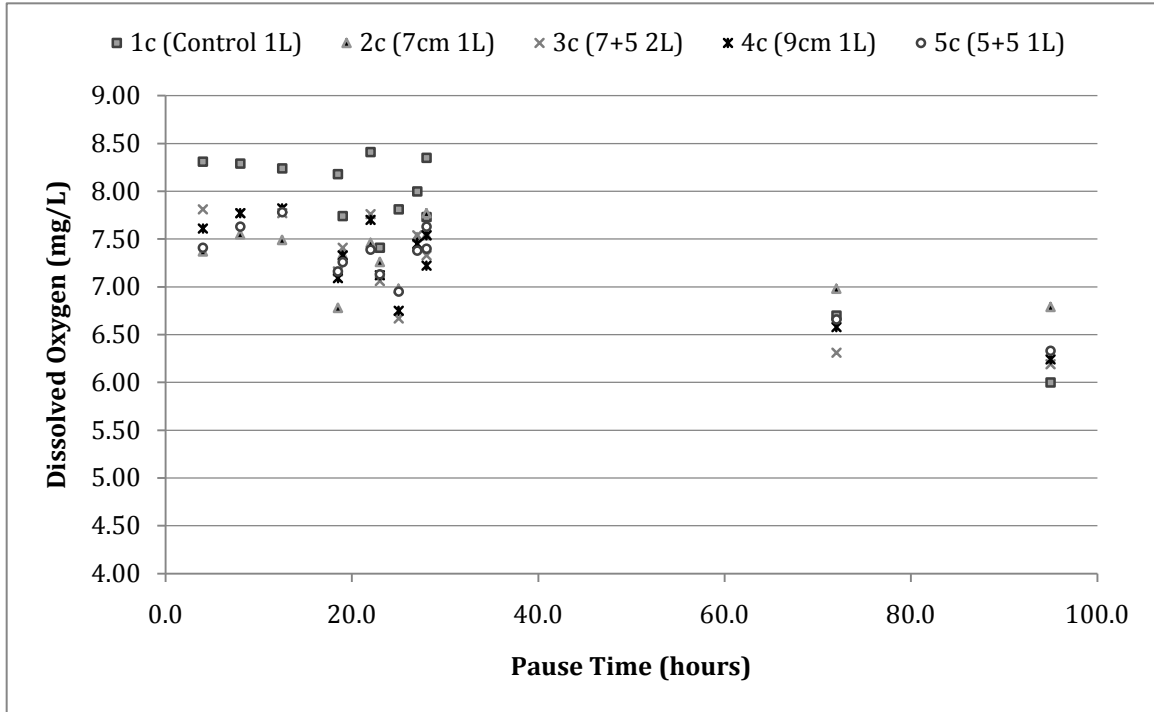


Figure 31 – Phase 3 Sample Port Dissolved Oxygen by Pause Time

Figure 31 and Figure 32 present the DO from phase 3 by pause time. Here, tests at a broad range of pause times are quite revealing, with an almost linear relationship and distinct patterns between filters. Filter 3c drops below, probably due to the higher flow volume providing more solids deposition and associated higher oxygen demand.

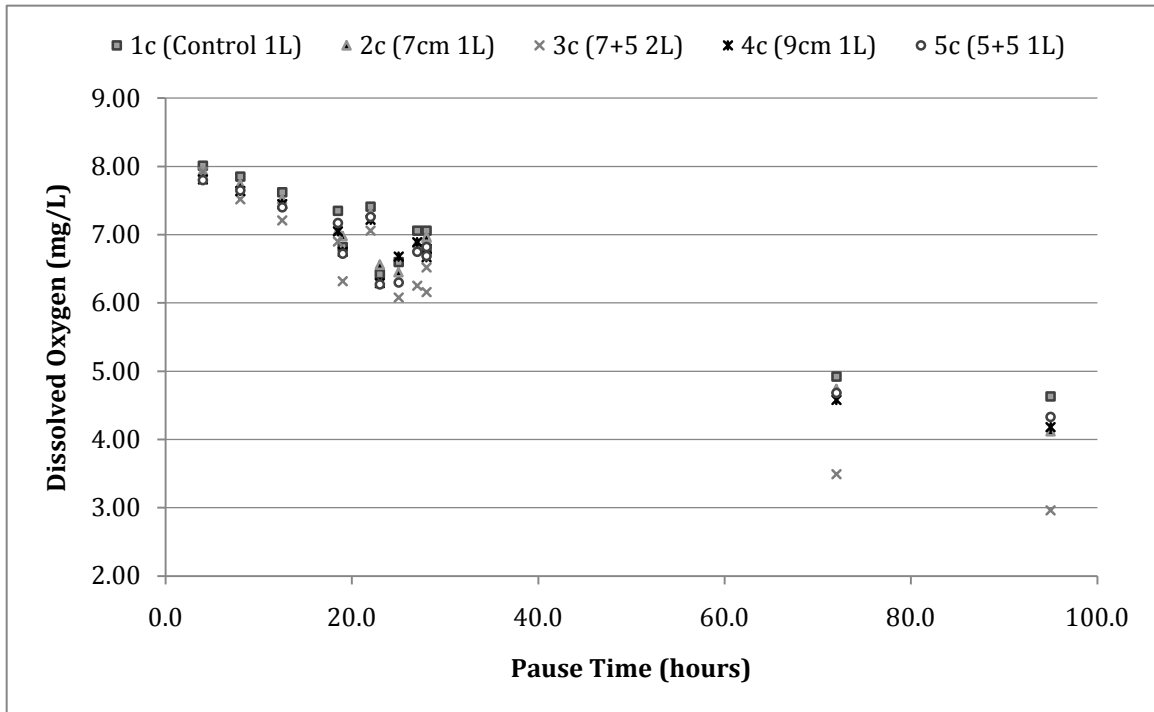


Figure 32 – Phase 3 Filter Effluent Dissolved Oxygen by Pause Time

4.2.5 Other Filter Operation Results

In addition to the regularly conducted tests, several other one-time tests were conducted to further characterize the filter performance. These include turbidity and DO profiles, cations, anions, pH, alkalinity, total solids, dissolved solids, suspended solids, and particle counts.

Turbidity and Dissolved Oxygen Profiles

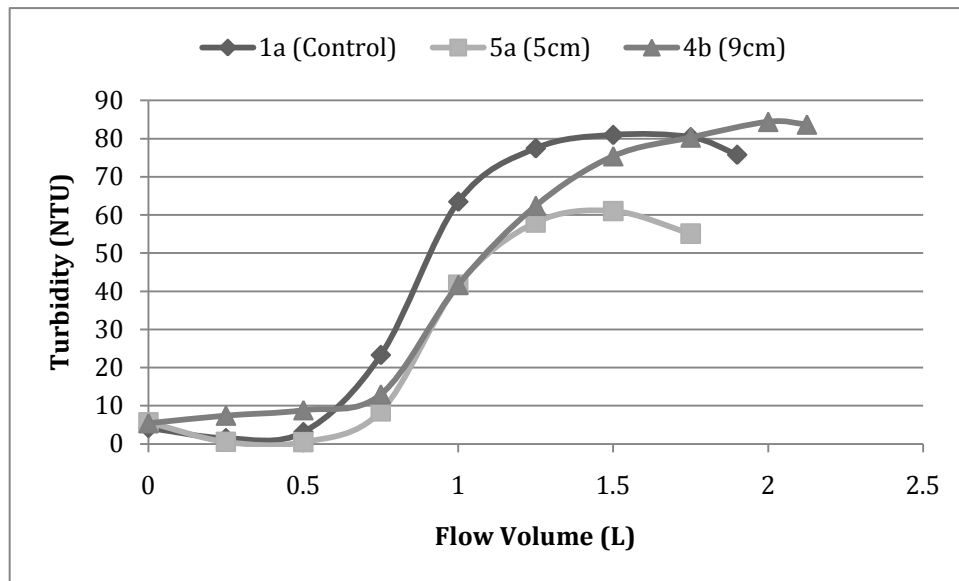


Figure 33 – Turbidity Profiles: Turbidity over Flow Volume

Figure 34 presents the same data as Figure 33, except with the turbidity presented as percent reduction and the flow volume standardized against the Pore Volume (PV) of that particular filter. The PV appears to align the curves. The low turbidity observed for filter 5a was measured prior to the first maintenance. Repetition of this procedure after maintenance would raise the peak to approximately the same as the other two. The slight decrease in turbidity after the peak suggests that dwindling flow rates slightly impact treatment efficiency.

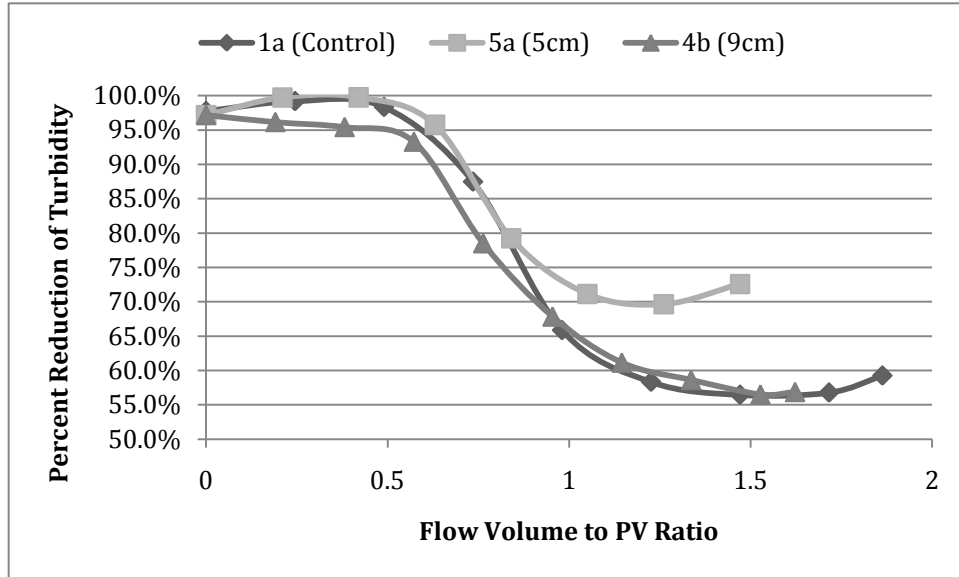


Figure 34 – Turbidity Reduction vs. Flow Volume

Figure 35 presents the only DO profile measured. The only conclusion suggested is that the DO is relatively low at about 0.75 PV. Unfortunately, due to complications with the DO probe (see discussion in Section 3.4.3), this test was set aside, and it was not until after the filters were decommissioned that it was observed that the test needed repeating. While this conclusion is consistent with theory and with the comparisons between the sample port and the filtrate, further data is necessary to experimentally verify it.

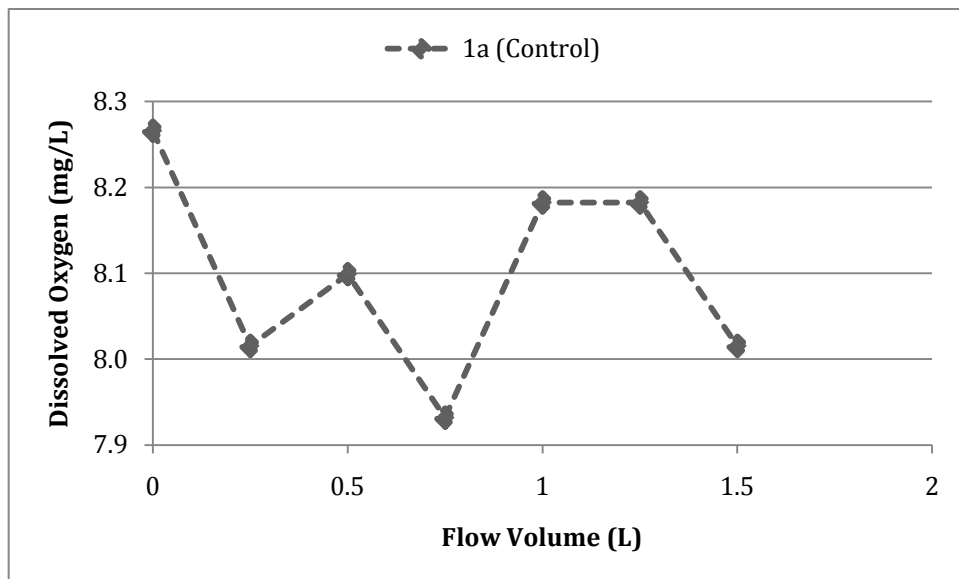


Figure 35 – Dissolved Oxygen Profile

Cations, Anions, pH, and Alkalinity

Further geochemical tests were conducted on feed water and filtrate on two occasions: Feb 11th and April 7th. The Feb 11th samples were collected just after turbidity measurements were taken for the day. On April 7th, a more representative sample was taken by mixing 0.5 L of the filtrate after 0.25 L had passed through. An additional sample from filter 3c was taken after 1.25 L had passed through, to compare the differences between different residence times.

Anion concentrations were determined using Ion Chromatography. Cation concentrations were determined using Atomic Absorption.

Table 14 – Feb 11 Anion Concentrations

	Effluent Volume	Fluoride (mg/L)	Chloride (mg/L)	Sulfate (mg/L)	Bromide (mg/L)	Nitrate (mg/L)
W.R. Water	N/A	0.044	23.611	7.610	0.014	0.368
Feed Water	N/A	0.045	23.247	7.542	0.014	0.362
1a (Control)	1.53 L	0.080	23.742	8.783	0.015	0.428
2a (1cm)	1.53 L	0.077	23.539	8.742	0.014	0.408
3a (3cm)	1.53 L	0.071	23.429	8.842	0.013	0.417
4a (3cm 3L)	1.53 L	0.060	16.463	5.906	ND	0.298
5a (5cm)	1.53 L	0.050	23.433	9.327	0.013	0.452

Table 15 – April 7 Cation Concentrations

	Test Volume	Fe	Ca	Mg	K	Na	Cu	Mn
Feed water	N/A	0.717	3.89	0.744	1.565	18.83	0.014	0.005
1c (Control 1L)	0.25 L to 0.75 L	0.107	4.34	0.920	1.998	18.32	ND	0.016
3c (7+5 2L)	0.25 L to 0.75 L	0.072	4.23	0.902	1.736	17.93	ND	0.025
3c (7+5 2L)	1.25 L to 1.75 L	0.419	3.67	0.800	1.734	17.83	ND	0.014
4c (9cm 1L)	0.25 L to 0.75 L	0.052	4.13	0.903	1.993	18.44	ND	0.023

Table 16 – April 7 Anion Concentrations

	Test Volume	Fluoride (mg/L)	Chloride (mg/L)	Sulfate (mg/L)	Nitrate (mg/L)	Phosphate (mg/L)
Feed water	N/A	0.062	29.025	11.192	0.889	0.126
1c (Control 1L)	0.25 to 0.75 L	0.080	29.275	12.814	1.402	ND
3c (7+5 2L)	0.25 to 0.75 L	0.068	29.142	12.888	1.381	ND
3c (7+5 2L)	1.25 to 1.75 L	0.075	29.350	12.512	0.934	0.124
4c (9cm 1L)	0.25 to 0.75 L	0.049	29.152	13.003	1.331	ND

Table 17 – April 7 pH and Alkalinity

	Test Volume	pH	Alkalinity (mg/L)	Estimated T-CO₂ (meq/L)
Feed water	N/A	7.15	8.38	0.193
1c (Control 1L)	0.25 L to 0.75 L	6.78	8.21	0.223
3c (7+5 2L)	0.25 L to 0.75 L	6.62	6.64	0.202
3c (7+5 2L)	1.25 L to 1.75 L	6.64	5.92	0.177
4c (9cm 1L)	0.25 L to 0.75 L	6.6	6.02	0.186

Solids Analyses and Particle Counts

On March 22nd, filters 1b and 4b were tested for solids and particles. Total suspended solids were measured by passing through a filter membrane and measuring the change in dry weight. Total dissolved solids were calculated by subtracting the suspended from the total.

However, the total suspended solids concentrations are suspiciously low; especially since the feed water was manufactured with 200 mg/L of kaolinite. This is most likely caused by outdated desiccant, allowing the membranes to reabsorb moisture from the air while cooling, prior to measurement. The estimated corrected concentrations are made assuming that the feed water concentration was approximately 200 mg/L, and that all membranes absorbed the same amount of moisture. The corrected TDS concentrations approximately match those expected based on the cation, anion, and alkalinity measurements above.

Table 18 – Total Solids

Sample	Effluent Volume (L)	TS (mg/L)
1b (Control)	0.57	127.5
4b (9cm)	0.57	91.5
1b (Control)	1.50	150.5
4b (9cm)	1.75	144.5
Feed Water	N/A	280.0

Table 19 – Total Suspended Solids

Sample	Effluent Volume (L)	Measured TSS (mg/L)	Estimated Corrected TSS (mg/L)
1b (Control)	0.57	6.0	86
4b (9cm)	0.57	-5.0	75
1b (Control)	1.50	9.0	89
4b (9cm)	1.75	4.0	84
Feed Water	N/A	120.5	201

Table 20 – Total Dissolved Solids

Sample	Effluent Volume (L)	Measured TDS (mg/L)	Estimated Corrected TDS (mg/L)
1b (Control)	0.57	121.5	41.5
4b (9cm)	0.57	91.5	16.5
1b (Control)	1.50	141.5	61.5
4b (9cm)	1.75	140.5	60.5
Feed Water	N/A	159.5	79.5

It is interesting to observe that all filtrate samples in Table 21 when compared to the feed water demonstrate 1 to 3 log removal of particles greater than 7 microns, but the number of particles less than 5 microns increased significantly enough to have approximately the same number of total particles. It would seem that larger particles have been retained by all filters, perhaps even breaking larger particles down into smaller particles. While the total number of particles in the

effluent is comparable to the influent, the total mass is clearly significantly smaller. This qualitatively agrees with Table 18.

Table 21 – Particle Counts

Filter No.	Effluent Volume	2-3 μm	3-4 μm	4-5 μm	5-6 μm	6-7 μm	7-8 μm	8-9 μm
1b (Control)	0.57 L	5,698.8	5,117.9	6,317.9	1,997.0	1,472.0	2,104.1	180.5
4b (9cm)	0.57 L	9,435.9	2,526.8	1,331.8	245.5	138.3	178.1	11.2
1b (Control)	1.50 L	7,731.8	6,991.4	7,172.0	1,733.3	1,044.3	1,154.6	74.4
4b (9cm)	1.75 L	9,131.7	7,528.4	6,219.5	1,130.4	560.4	515.4	27.4
Feed Water	N/A	801.2	776.9	1,642.1	1,055.1	1,303.8	5,063.9	930.0
<hr/>								
Filter No.	Effluent Volume	9-10 μm	10-12 μm	12-15 μm	15-20 μm	20-25 μm	25-100 μm	2-100 μm
1b (Control)	0.57 L	433.4	430.9	139.0	172.7	8.3	3.4	24,075.7
4b (9cm)	0.57 L	27.7	30.2	13.6	33.4	4.2	2.0	13,978.6
1b (Control)	1.50 L	152.6	116.5	33.3	45.4	3.7	1.3	26,254.5
4b (9cm)	1.75 L	52.4	38.5	11.9	11.8	0.7	0.3	25,228.7
Feed Water	N/A	2,912.3	4,507.1	3,021.3	5,200.4	442.0	131.5	27,787.4

4.3 Targeted Observations

In this section, the raw data presented above are analyzed and specific trends are highlighted. Particular emphasis is given to the impacts of the specific design parameters of the DSL modification. Several of the trends observed point to the importance of the operational parameters which users are trained to follow; the impacts of several of these parameters are also highlighted. The behavior of the filter in response to variations in feed water quality is discussed. Section 4.3.4 discusses whether the DSL is an improvement upon the BSF design for various feed water conditions, and the final section discusses the applicability of the results of this experiment to field conditions at full-scale.

4.3.1 DSL Configuration

Three basic parameters regarding the configuration of the Dual Sand Layer were experimentally isolated and observed: the thickness of the supernatant water layer; the thickness of the fine sand layer; and the presence of a coarse sand layer supporting the fine sand layer. Each of these parameters are considered in the following three subsections.

Additional parameters that were not experimentally examined were still carefully considered, and are discussed in subsection 4.3.1.4. These include the thickness of the intermediate water layer, the density of holes in the DSL stand which supports the sand, and the design of the DSL stand, specifically whether a direct water path to the atmosphere is included.

4.3.1.1 *Supernatant Water Thickness*

Two pairs of filter configurations varied only by the supernatant water layer thickness: filters 2b and 3b, and filters 3b and 3c. Two pairs also varied by only two parameters, including supernatant water thickness: filters 2c and 5c; and filters 2c and 3c. By comparing these pairs of filter apparatuses, and other general observations, the supernatant water thickness can be assessed. These observations are summarized in the final paragraph of this subsection.

Filters 2b and 3b had respective thicknesses of 1 cm and 3 cm. Unfortunately, the sample port clamp for filter 2b broke on March 1st, draining the DSL and leaving it dry for a day, disrupting the oxygen equilibrium for the next few days. However, by March 8th, the differences between filters 2b and 3b in terms of DO in the sample port are negligible. The turbidity, *E. coli*, and flow rate are also generally quite similar. The one discernable difference between the two filters is that the DO in the filtrate tends to drop a little farther (0.2 to 0.5 mg/L) for days with high pause time (greater than 30 hours).

Filters 3b and 3c had respective thicknesses of 3 cm and 5 cm. The differences in feed water quality between the two filters mask any other observable changes, but there does appear to be slightly lower DO (by 0.5 mg/L) in the filtrate associated with the thicker water layer.

It is interesting to observe that filters 4c and 5c had the same total thickness, with 4 cm of sand in filter 4c replaced by 4 cm of water in filter 5c. The DO results were remarkably similar between the two, indicating that total thickness has more impact on DO than whether the thickness is composed of sand or water.

Based on the above comparisons, it appears that the total thickness above the *schmutzdecke* correlates with lower dissolved oxygen concentrations, especially in the filtrate at pause times greater than 24 hours. The same trend is observed whether that thickness consists of sand or water. Based on this observation, it would appear that in the filter apparatus, DO transfer is probably occurring primarily in the 1/8" thick direct water channel surrounding the DSL stand.

Gouging of the DSL was observed for all modified filters, even with 5 cm thick supernatant water. For filter 3c, the diffuser was actually at about 4 cm, or 1 cm beneath the water level. For filter 5c, the diffuser was about 1.5 cm above the water layer, so that when the water rises during operation, jets of water pushed through the diffuser, gouging the sand 6.5 cm beneath. The initial intent in the laboratory design was not to avoid gouging, which would keep the top sand layer from forming a ripe *schmutzdecke*, potentially depriving the lower sand layer of oxygen. However, gouging appears to also disrupt particle removal, and should therefore be avoided. This can be avoided by either increasing the water thickness to 5 cm and the height of the diffuser to above 7 cm, or by increasing the hole density in the diffuser without raising the diffuser too high.

In summary, three primary points are highlighted:

- It is observed that DO appears to transfer primarily through the direct water channel surrounding the DSL stand.
- Greater thicknesses of sand and water are associated with lower DO (by as much as 1 mg/L), especially in the filtrate and especially at high pause times.
- Gouging of the DSL should be avoided by increasing the supernatant water thickness to 5 cm and appropriately designing the diffuser.

4.3.1.2 *Dual Sand Layer Thickness*

The thickness of the DSL was one of the primary variables in the laboratory experiments. In each phase, several different thicknesses were tested, ranging from as low as 1 cm (filter 2a) to as high as 9 cm (filters 4b and 4c). In many cases, the DSL thickness was the only parameter varied; in others, multiple parameters were varied. However, little correlation between the sand thickness and treatment efficiency is observed, and DO appears to be bypassing the sand altogether.

Greater thickness clearly correlates with lower flow rates. This is intuitive; more sand means more head loss and lower flow rates. However, in the filter apparatuses, more sand also means less freeboard and a higher effluent spout, which also drops the flow rate. So the change in flow rate is only partially due to the greater sand thickness.

This lower flow rate appears to have a more profound impact on the filter performance than the sand thickness itself. In general for individual filters, the turbidity reduction increased over a period of weeks as the filter clogged and the flow rate dropped. More relevantly, for all turbidity profiles, the turbidity of the filtrate was observed to peak at about 1.5 pore volumes and then drop slightly. This slight drop in turbidity at the end of filter operation appears to be due to the decreased flow rate, which increases the residence time of the water within the filter pore spaces.

Therefore, although slight improvement in turbidity reduction at higher DSL thicknesses is observed, the extent of this improvement is easily within the range explainable by flow rate reduction. Additionally, the lower initial flow rate compounded by the higher deposition rate helps to explain the trend that filter maintenance tended to occur more quickly for filters with greater sand thickness.

As observed in Section 4.3.1.1, DO flux appears to be associated with the total thickness, combining both sand and supernatant water, rather than with the thickness of either parameter independently.

It should be noted that the pits observed by gouging were consistently 2 to 3 cm deep. While gouging should be avoided by proper design of the supernatant water layer and the diffuser, the

DSL if used should also be greater than 3 cm to ensure that gouging does not completely short circuit the DSL.

4.3.1.3 Observations on the Coarse Sand Layer

The only filter configuration that lacked a coarse sand layer beneath the fine sand in the DSL was filter 2a, or filter 2 during phase 1. The purpose of the coarse sand layer is to hold the fine sand layer in place, and keep it from leaking through the holes in the DSL stand base. Filter 2a was included as a configuration to determine whether this is necessary. It was initially suspected that, if the coarse sand was not necessary, then a thicker fine sand layer would be obtainable without reaching anoxic conditions, which might in turn increase the treatment efficiency of the filter.

Based on the performance of filter 2a, it is clear that the coarse sand layer is necessary, or that it must at least be replaced by something equally effective. Based on the figures in Section 4.2.1, filter 2a consistently performed very poorly. The effluent turbidity of filter 2a is the highest almost without exception. The dissolved oxygen was also generally the lowest, which is remarkable since the total thickness of the water column above the *schmutzdecke* was the least among all the filters. This does not appear to be explained by more efficient microbiological treatment, since filter 2a has either the highest or 2nd highest *E. coli* concentration in six out of eight points.

It is interesting to observe that the flow rate of filter 2a decreased initially, and then appeared to reach a stable plateau. Similar behavior was observed in filter 1a (the control filter), although with a significantly lower flow rate overall. Filter 1a was not reconfigured at the end of phase 1, and the flow rate eventually kept dropping until filter maintenance was required. Undoubtedly this would have occurred for filter 2a as well, but over a longer time span.

In addition to the observations based on the collected data, it was observed that sand did in fact pass through the DSL stand base, forming heaps on the lower sand layer. These heaps and their associated increase in surface area might have caused the lack of clogging. In addition, the heaps might have served as obstacles, limiting the mixing of water during the pause time, and therefore

inhibiting the transfer of oxygen. This would explain the low dissolved oxygen concentrations measured.

4.3.1.4 Other Design Parameters

In addition to the DSL design parameters that were included in part of the filter configurations, several other parameters were considered. In particular, the thickness of the intermediate water layer and the density of holes in the DSL Stand were carefully considered to facilitate healthy growth in the *schmutzdecke*.

The intermediate water layer thickness and the DSL Stand hole density were examined during preliminary design and testing of the laboratory filter apparatuses. The intent was to provide the minimum possible thickness while still permitting complete mixing to occur, without gouging the sand layer by rapid jets of water from above. A target of 1 cm was initially proposed. The hole density in the DSL thickness was increased incrementally, setup 1 cm above a bed of sand, and water was poured through the DSL. At low hole densities, gouging was rapidly visible. As the hole density was increased, the flow rates decreased, until gouging was eliminated. A hole density of 11 holes in 62.1 cm² was selected. The holes were 2 mm in diameter. No problems with this density were observed throughout the course of experiments.

As discussed above, it appears that oxygen transfer is primarily occurring through the direct water channel around the perimeter of the DSL. In designing the laboratory filter apparatuses, it was recognized that this direct channel would exist and several options for sealing the gap were considered. Any such feature would have been difficult to implement, and would have greatly hindered the reconfiguration of the filters and the cleaning of the lower sand layer in operation. However, the air flow at the top of the apparatus was intentionally hindered by the system of tape and wires on the inside of the filter tube and on the outside of the DSL tube which support the DSL. In addition to this, a cap was placed on top of the filter apparatus.

It was recognized that this same challenge would occur for scaling up the filter to the household scale: in all likelihood this water channel would exist and be significantly larger, especially if the DSL is designed to be removed so that the lower layer can be maintained as necessary.

Therefore, this direct water channel was permitted in the laboratory design. Specific recommendations on how to design this aspect of the full-scale filter are included in chapter 5.

4.3.2 Operational Scheme

Based on the data gathered in this research program, it appears that appropriate operation is of critical importance for maximum treatment of high turbidity water. Regardless of whether the BSF includes a DSL, the filter must be sized and configured in a way that is consistent with user needs, and the users must be trained to operate the filter in a way that is consistent with the filter design. The run cycle water volume, the pause time between run cycles, and filter maintenance all have a significant impact on the filter performance. These conclusions and their supporting data are described in detail in this section. Specific recommendations which further apply these conclusions are discussed in chapter 5.

4.3.2.1 *Run Cycle Water Volume*

The run cycle water volume is perhaps the most important operational scheme parameter. The trend clearly indicates that the turbidity dramatically increases as the flow volume approaches and surpasses the pore volume. Well above the pore volume, increases in flow volume result primarily in increased clogging, and the associated side effects.

This pattern is observed in several cases. The clearest example is a series of turbidity profiles of phase 1 and phase 2 filters. The pattern is also clearly demonstrated in phase 3 by noting the relationship between pore volume and turbidity reduction of each filter.

In phase 1, filter 4a was configured physically identical to filter 3a, with the only difference being a 3 L flow volume in filter 4a, compared with the standard 2 L flow volume which was used in the rest of the filters. In addition, during phase 3 all but filter 3c were operated at 1 L per day. Since the pore volume for each filter varied, the flow volume to pore volume ratio ranged from 98% for filter 1c to 76% for filter 4c. In addition,

Figure 34 is duplicated here, presenting the turbidity profiles. Note that filter 5a was tested during the unique period before the first cleaning where filter 5a demonstrated remarkably high

turbidity reductions even before the flow rate dropped significantly; so the point at which the turbidity levels off is much higher than that observed in all other cases.

Nevertheless, the pattern is clear: at less than 0.5 PV, the percent reduction of turbidity is greater than 95%. Soon after 0.5 PV, the treatment efficiency begins rapidly dropping. The rate of drop appears to slow down around 1.0 PV, to eventually level off between 1.25 and 1.5 PV. It appears that the first 0.5 PV to exit the filter consists entirely of water that has been retained in the filter for the duration of the preceding pause time. After 1.5 PV, all of the water from the pause time has been flushed through, so that 100% of the effluent at this point was placed into the filter and immediately flushed through. Between 0.5 and 1.5 PV, it appears that some mixing is occurring between the two categories.

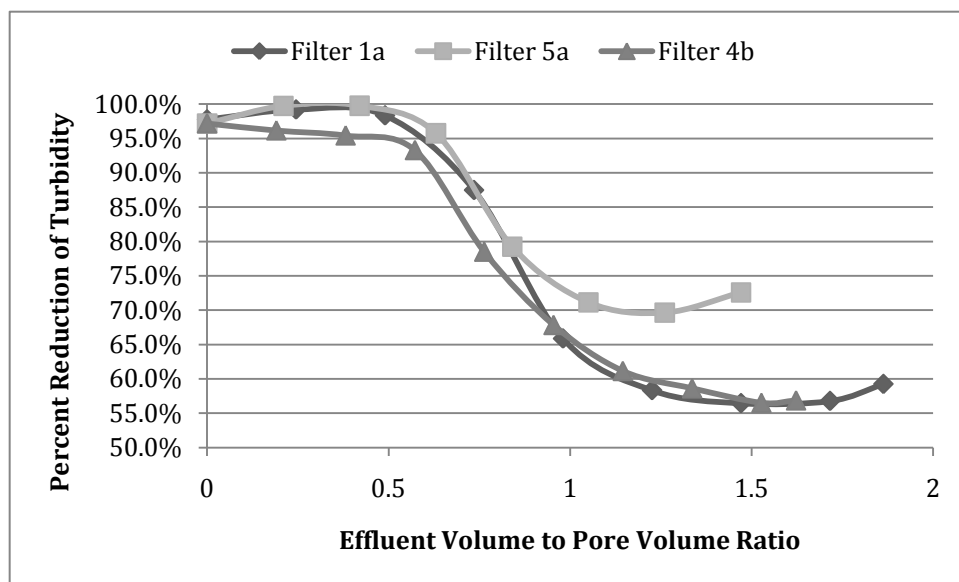


Figure 34 – Turbidity Reduction vs. Flow Volume (Reproduced from page 77)

Figure 36 (below) presents the turbidity reduction of Phase 3 as a function of pause time. During Phase 3, the turbidity of most filters was taken at 0.7 L, which was between 53% and 69% of the pore volume. Figure 36 clearly indicates that turbidity reduction is positively correlated with two variables: increased pause time (discussed in Section 4.3.2.2); and decreased pore volume. At all pause times, a significant improvement in treatment was seen between filter 2c at 0.56 PV, and filter 1c at 0.69 PV. This corroborates the observations from Figure 34 that the turbidity reduction drops sharply after about 0.5 PV.

On the other end of the scale, several observations can be made regarding very high flow volumes. Filters 3a and 4a differed only in their flow volume, which in both cases was significantly higher than the pore volume. It appears that the primary result of the increased flow volume was an increase in the accumulation rate of solid mass on the *schmutzdecke*, which introduced higher head losses and decreased the flow rate. The more quickly decreasing flow rate triggered filter maintenance sooner. However, due to the timing of phase 1, both filters underwent filter maintenance only once. All other things being equal, it appears that the filter cycle is approximately inversely proportional to the flow volume. This same observation would be expected for cases where small volumes are being filtered several times in a day resulting in total daily flows comparable to filters 3a and 4a.

It should be noted that turbidity and *E. coli* tests were conducted after 1.5 L had passed through both filter 3a and 4a. Based on the turbidity profiles presented in Figure 34, it is probable that the turbidity measured is representative of the effluent after approximately 1.25 pore volumes had passed through. Therefore, the overall average turbidity of the effluent for filter 4a is significantly higher than that of filter 3a, since a much greater percentage of the flow is characterized by this peak turbidity.

4.3.2.2 *Pause Time*

The residence time within the filter appears to be the most important parameter for high turbidity conditions. Even slowing down the flow rate to increase the residence time from 7 minutes to 10 minutes makes a consistent measurable reduction in the turbidity. Increased treatment efficiencies are documented at every stage from a minimum of 7 minutes to almost 100 hours. At the same time, higher pause times are associated with greater DO drops that are almost linear with time. It appears that steady state DO flux was not reached within 100 hours.

Effect on Turbidity

The longer a particular unit of water is within the filter, the greater the turbidity reduction can be expected. This is true on the short end of the spectrum, as well as the long end. This section demonstrates this observation starting at 7 minutes and then proceeding to longer times, approaching 100 hours.

In Figure 34 the turbidity profile of several filters are presented. As discussed in Section 4.3.2.1, the final leveling off of the turbidity reduction after about 1.25 pore volumes indicates that at this point, 100% of the filter effluent consists of water which was poured through minutes earlier. The feature of interest here is the slight increase observed in all three profiles at the very end of the run cycle.

Due to the hydraulic control of the effluent spout, the instantaneous flow rate for a run cycle is at the maximum within the first few seconds of operating the filter. For filters with greater than 1 L cycle volumes, the flow rate was initially held constant by topping off the water level until the complete cycle volume had been poured into the filter. After this, the flow rate then decreases gradually as the water level within the filter approaches the height of the effluent spout. By the end of the run cycle (the last few 100 mL, at the laboratory scale), the flow rate is a small fraction of the initial flow rate.

Going back to Figure 34, the only difference between the filtrate after 1.5 PV and the filtrate after 1.75 PV was that the flow rate dropped significantly. This extended the residence time of the water within the filter. Although the instantaneous flow rates were not measured, it is estimated that at 1.5 PV the residence time was approximate 7 minutes, while at 1.75 PV, the residence time was a few minutes more, perhaps as much as 10 minutes. This small change resulted in a slight but measureable increase in turbidity reduction of a few percent.

Figure 36 presents the turbidity reduction of pore volumes from 4 hours to 95 hours. The data plotted here is taken from phase 3, so that the pore volume could be presented simultaneously (denoted by the % labels, discussed in Section 4.3.2.1). The points plotted at 20.6 hours and 27.0 hours are based on averages, each incorporating four run cycles. The data for Filter 3c (140% PV) are included as a standard to compare the feed water quality behavior, since the water at 1.4 PV has a residence time of only a few minutes.

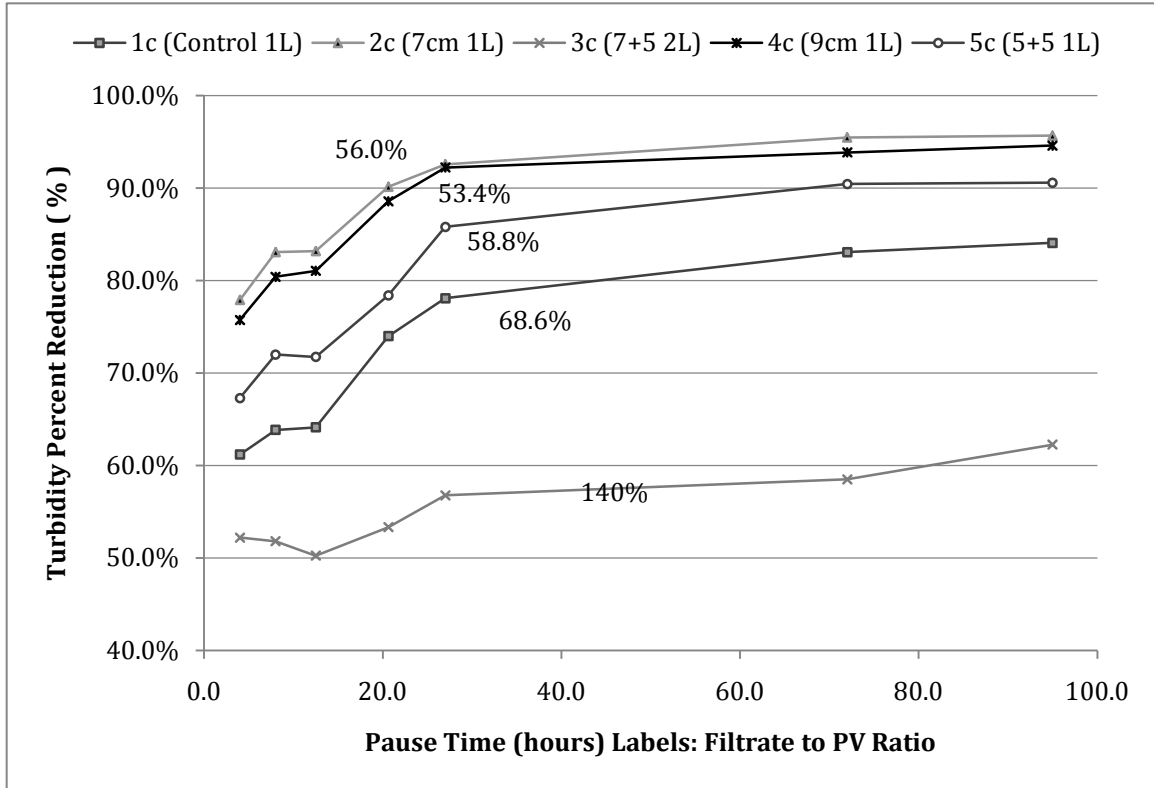


Figure 36 – Turbidity Reduction Dependence on Pause Time and Pore Volume

The trend is quite clear: at higher pause times, the turbidity reduction increases. The change is rapid at a few hours, and levels off gradually after 24 hours. The flat behavior at 12 hours is probably due to feed water quality, observing the dip in the filter 3c curve. Between 24 and 100 hours, differences in feed water quality cover up any potential trends.

Effect on Dissolved Oxygen

Out of all the parameters that impact dissolved oxygen concentrations, it appears that pause time has the most profound and important effect. Specifically, greater pause times consistently result in lower DO concentrations, especially in the filtrate but also in the sample port.

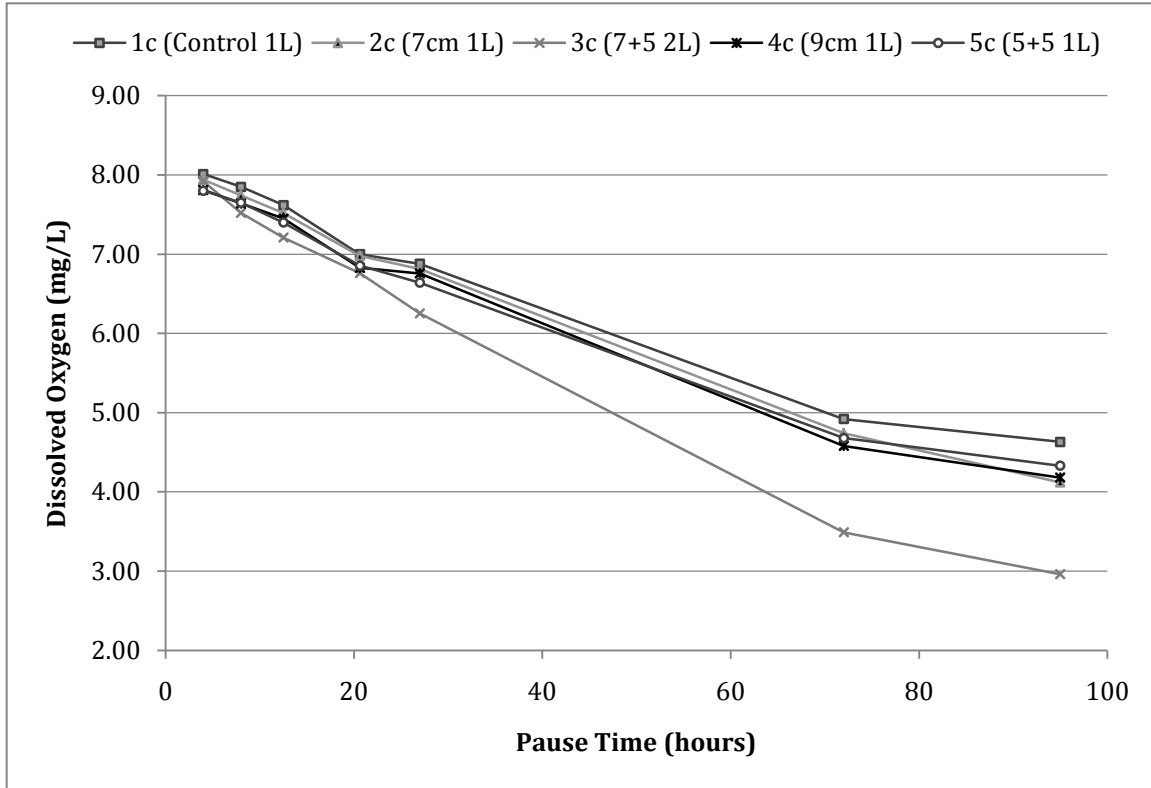


Figure 37 – Filtrate DO Correlation with Pause Time

First, it appears clear that the filtrate DO is consistently lower and more critical than the sample port DO. Based on this observation, DO from the filtrate will be used for the following analyses, even though most observations will apply generally to both. This phenomenon could be due to several reasons, hypothesized below. Unfortunately, no further data are available to corroborate these suggestions.

- DO demand from the *schmutzdecke* creates an oxygen draw extending both upward and downward. The DO above can be replenished by molecular diffusion extending up to the air surface, while the DO below has no option for replenishment. Therefore, the DO drops more quickly below than it does above.
- The biofilm growth actually extends farther than merely 2 cm into the sand, perhaps extending as far as 5 or 10 cm. The lowest DO levels would be expected at the center of the layer demanding DO. Perhaps the filtrate at 0.75 L consists of primarily water which resided at the center of the *schmutzdecke* during the pause time, resulting in the lowest possible DO concentrations.

- Perhaps the sample port mechanism allows too much mixing to occur during sampling, such that the measured water sample from the port consists of some water from right above the *schmutzdecke* and some water from the water channel which reaches toward the water surface.

It is interesting to observe the almost linear drop in DO with increased pause time, for each filter (Figure 37). The shortest pause time measured was 4 hours, with DO concentrations at approximately 7.75 mg/L. After 95 hours, the DO dropped to between 3 and 4.75 mg/L. Apparently steady state is not reached within 72 hours, because the DO continues to drop afterward. The longest pause time observed was 95 hours, and the DO for filter 3c, with a total thickness above the *schmutzdecke* of 14 cm reached 2.96 mg/L.

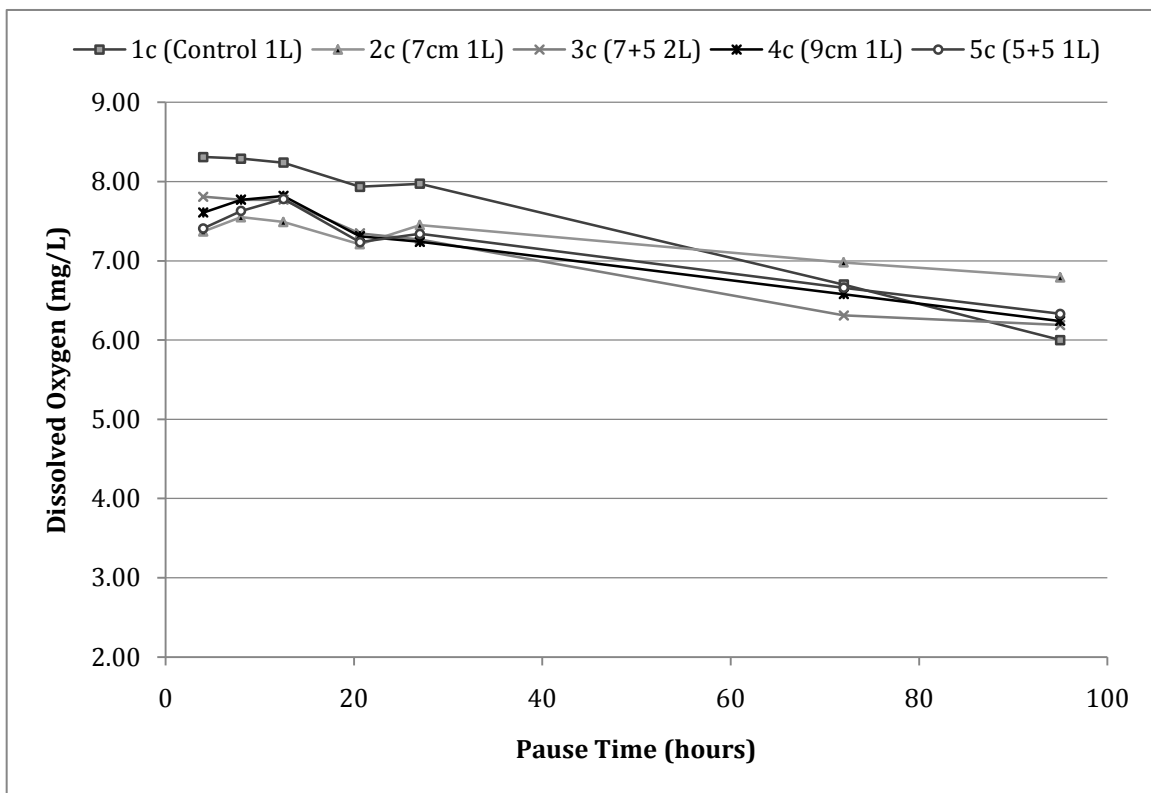


Figure 38 – Sample Port DO Correlation with Pause Time

While Figure 37 is plotted for phase 3, the observation holds true in general for all of the data collected. With one curious but extraneous exception, DO never dropped below 6 mg/L with pause times less than 30 hours. The one exception was the strange behavior observed after filters

2, 3, and 4 were reconfigured from phase 1 to phase 2. During this time, high turbidity reductions, high *E. coli* reductions, and remarkably low DO concentrations were all prevalent.

The differences between the filters appear to be relatively insignificant compared to the pause time. Filter 1c is unmodified, with a supernatant water layer of 5cm. While this filter has the highest DO at every point, even the lowest filter after 50 hours has a higher DO than filter 1c after 95 hours.

Nevertheless, between the four filters with the same flow regime, the DO appears to correlate with the total thickness of sand and water above the *schmutzdecke*. Filter 3c DO is noticeably different from the other four. This could be due to two mechanisms, both stemming from the fact that filter 4c differs from the others in that the daily volume is 2.25, instead of 1 L. The first and more likely potential mechanism is that larger amounts of particulate matter is deposited within the filter, providing higher amounts of organic matter to be metabolized, resulting in a higher DO demand. The second potential mechanism is that higher clogging might physically restrict DO transfer from reaching the lower portion of the *schmutzdecke*.

4.3.2.3 Filter Maintenance

Determining the impact of maintenance (cleaning) on filter performance is important. After discussing the expected results based on the literature, this section describes the actual impacts observed. Namely, significant turbidity peaks as well as general turbidity increases of about 12% were measured after maintenance. *E. coli* trends proved difficult to quantify, but a decrease of approximately 8% *E. coli* removal after filter maintenance was measured.

Table 22 – Maintenance Frequency

Filter	Operational days between Filter Maintenance Procedures (days)		
	1 st Cycle	2 nd Cycle	3 rd Cycle
1a/b (Control)	64	N/A	N/A
2a (1cm)	49 ¹	N/A	N/A
3a (3cm)	28	21 ¹	N/A
4a (3cm 3L)	37	12 ¹	N/A

5a/b (5cm)	27	17	20
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Note: The physical configurations of filters 1 and 5 were not changed between phases 1 and 2.

Expected Results Based on the Literature

According to the literature on SSF, filter maintenance can result in significantly reduced treatment efficiency both in terms of turbidity and microbiology. In fact, the reason behind the general recommendation that the BSF not be used for high turbidity water is a combination of two established facts: filter maintenance hinders treatment; and high turbidity waters result in frequent maintenance.

However, the fact that filter maintenance hinders treatment does not appear to be well-documented. Stevenson (2008) suggests that research on the relative impacts of different cleaning mechanisms is lacking and necessary. He also cites a personal conversation with the presenter of a poster presentation by Jenkins *et al.* (2008), asserting that “cleaning the unit on a schedule or too frequently disrupts effective treatment”. Unfortunately, the poster itself does not discuss filter maintenance.

Observed Effect on Flow Rate

The purpose of maintenance is to remove clogging particles in order to restore the flow rate. Accordingly, flow rate increases were always observed to occur after regular filter maintenance (see Table 23). It is interesting, however, to observe the relative change in flow rate between successive maintenances. The only filter to be maintained multiple times without reconfiguration was filter 5. Filter 5 was maintained 3 times at approximately 20 day intervals. The flow rate the two days after maintenance was successively 0.167 L/min, 0.155 L/min, and then 0.145 L/min. This pattern is probably indicative of clogging on the lower sand layer, which was never maintained. Based on this pattern, it is estimated that maintenance on the lower sand layer would be necessary after approximately four to six months.

However, it has already been observed that some depth filtration is occurring; treatment continues during the pause time and does not merely occur when the water passes through the *schmutzdecke*. Therefore, some accumulation of settled particles must be occurring at depth

within the sand bed. Eventually, this will inevitably result in head losses that are not recovered by cleaning only the top 1 cm of sand. Based on the similarity between the flow rates after maintaining filters 5 and 1 on 3/17, this may be what is actually occurring in both filters, rather than accumulation on the lower sand layer for filter 5.

Table 23 – Filter Maintenance Effects on Flow and Turbidity

Filter No.	Maintenance Date	Average Flow Rate (L/min)		Turbidity (NTU)		
		2 Days Before	2 Days After	Day Before	1 st Day	2 nd Day
5a	1/26	0.048	0.167	57.5	160	105
4a	1/27	0.108	0.173	93.5	130	101
3a	2/8	0.110	0.164	109	137	109
5a	2/17	0.112	0.155	93.0	122	108
2a/b ¹	2/24	0.169 ¹	0.153 ¹	114 ¹	N/A ¹	75.4 ¹
3a/b ¹	2/24	0.160 ¹	0.149 ¹	111 ¹	N/A ¹	63.4 ¹
4a/b ¹	2/24	0.132 ¹	0.136 ¹	104 ¹	N/A ¹	58.7 ¹
1b	3/17	0.099	0.149	95.2	143	109
5b	3/17	0.085	0.145	87.9	125	107

Note: 1) See 4.2.2 for discussion on the behavior of filters 2, 3, and 4 after reconfiguration for phase 2.

Observed Effects on Turbidity

In this laboratory program, maintenance was clearly observed to impact effluent turbidity measurements. Two effects were observed: a sharp peak the day immediately following maintenance, and a moderate long-term increase. Figure 39 illustrates the turbidity effects documented in Table 23. The long term % increase was calculated by dividing the turbidity on day 2 by the turbidity the day before. The short term % peak was calculated by dividing the day 1 turbidity by the day 2 turbidity. Therefore, the long-term peak should be understood as adding on top of the short term increase.

In Figure 39, it is interesting that the first maintenance of filter 5a resulted in such a large increase, both long term and short term. The long term increase is due to the remarkably low

turbidities observed prior to the first maintenance. The flow rate was allowed to drop significantly below the normal cutoff criteria, but even before that the turbidity reduction rates were higher than all the others observed in this research program. After maintenance, the turbidities observed in filter 5 were typical of those observed for all other filters.

The short term peak on 1/26, however, is more explainable. Filter maintenance consists of several iterations of stirring, decanting, and adding clean feed water. On that day, there was not enough feed water available for enough iterations to bring the supernatant turbidity to approximately equal with the feed water. With experience, the successive maintenance procedures observed a lesser peak, particularly for the other DSL filters (all except 1b). This illustrates that maintenance technique is important for the finished water quality; certainly for turbidity and most likely also for *E. coli*.

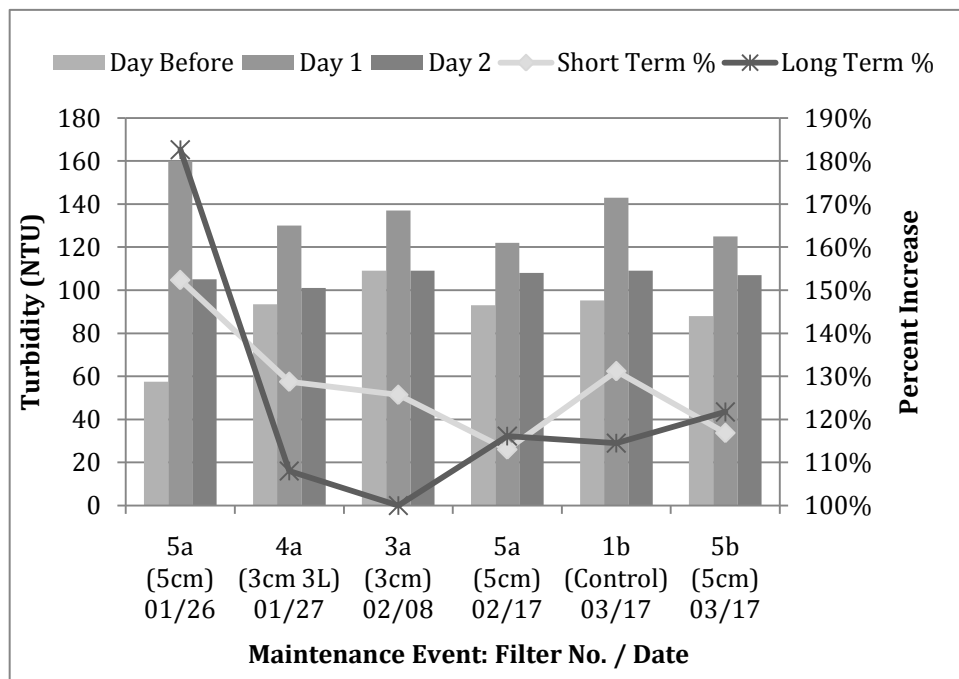


Figure 39 – Maintenance Effects on Turbidity

The final important observations from Figure 39 pertain to the differences between the unmodified filter 1 and the rest of the modified filters. As mentioned above, the short term peak tends to drop as time goes on due to improved maintenance technique. However, the short term peak for filter 1b is significantly higher than the peak for filter 5b, which was maintained on the

same day. It appears that the lower sand layer is serving to catch some of the particles released in the short-term peak. This is a benefit of the DSL.

The long-term increase observed in the unmodified filter is about the same as that observed in filter 5 on 2/17 and 3/17, and it is slightly worse than that of filters 3a and 4a. Therefore, the presence of the DSL appears to have no observable impact on the long-term response of the filter to maintenance. This result may only apply to the first few months of filter operation. It is possible that as the lower sand layer begins to clog over a period of several months, it will dampen the long-term increase observed.

Observed Effects on *E. coli*

Filter maintenance was observed to decrease the *E. coli* reduction capacity of the filters by approximately 10%. In this case, no distinction was found between modified and unmodified filters. This trend is definitely measurable and relatively consistent, but is actually small compared to day to day changes that do not have any easy explanation.

Due to the highly variable nature of *E. coli* concentrations and percent reductions, quantifying the effects of maintenance presented quite a challenge. Figure 40 illustrates the challenge and points to a logical and straight-forward method of quantification.

The data presented in Figure 40 consists of all five normal maintenance events for which a complete set of data is available. *E. coli* data does not exist prior to the first maintenance of filter 5a. Filters 2, 3, and 4 were also maintained and reconfigured between phase 1 and 2, but these were not due to flow criteria and the reconfiguration of the filters further complicates those data.

The columns in Figure 40 come in pairs; for each pair the left column represents the “Last” *E. coli* measurement prior to maintenance, while the right column represents the “Next” measurement after maintenance. The solid bar in each column denotes the percent reduction for the maintained filter. The vertical line indicates the range of percent reductions measured in all the filters that day, while the horizontal line marks the average percent reductions of all unmaintained filters.

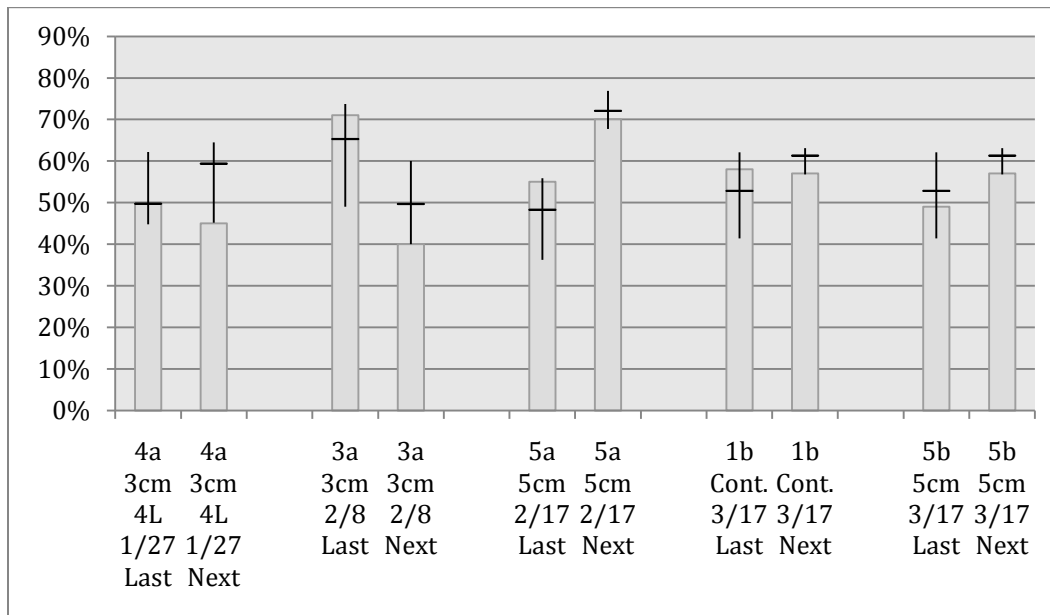


Figure 40 – Effects of Maintenance on *E. coli* Reduction

Simply comparing the percent reduction before and after maintenance would lead to erroneous results by neglecting to factor in variations in the feed water quality and the *E. coli* viability. For example, the maintenance of filter 1b resulted in almost no change of *E. coli* reduction numerically; but when compared to the rest of the filters' performance, filter 1b went from 5% above average to the lowest of the day at 4% below average.

Qualitatively, the effect of maintenance on *E. coli* reduction is apparent. After maintenance, the maintained filter almost always has the lowest concentration of the day. Prior to maintenance, the filter is above average in 3 cases, and only below average in 1 case. This can be quantified by calculating the difference between the filter and the average for each day, and comparing the day before with the day after. The effect of maintenance is calculated according to the following equation and summarized in Table 24.

$$(C_{Filter} - C_{AVG})_{NEXT} - (C_{Filter} - C_{AVG})_{LAST}$$

Based on this method of quantification, filter maintenance of the DSL filters ranges from 0% to 15% loss in *E. coli* reduction ability, with an average of 10%. The unmodified control filter is

right at the average for the modified filters, at 9%. It was initially postulated that maintaining the upper sand layer without disturbing the lower sand layer would have no impact on the microbiological quality of the filtrate, since the *schmutzdecke* would be undisturbed and functioning properly. This does not appear to be the case. The addition of a DSL does not appear to protect the *schmutzdecke* from being disturbed by filter maintenance.

For comparison, on days when *E. coli* concentration was measured without being immediately after any filter maintenance occurred, the difference between the maximum and minimum percent reduction was always at least 20%. If the same method of quantifying the effect of filter maintenance were used for days when maintenance did not occur, the range would be from +20% to -20%, with an average of 0% and a standard deviation of about 9%.

It appears that microbial treatment of the filters depends on many factors, with few readily observable patterns. One such pattern is that filter maintenance drops the microbial treatment by about 10%, but drops or jumps of 10% are very typical, often with no apparent explanation. Furthermore, the modified and unmodified filters are indistinguishable in this analysis.

Table 24 – Effect of Maintenance on *E. coli*

Filter	Date	Effect of Maintenance
4a	1/27	-15%
3a	2/8	-15%
5a	2/17	-9%
1b	3/17	-9%
5b	3/17	0%

4.3.3 Variations in Water Quality

In the field, water quality can vary significantly, potentially having disastrous effects on treatment technologies. In establishing any treatment technology, it is important to anticipate potential variations in water quality and determine the ability of the technology to respond. In this study, the chemistry of the water itself was not adjusted, since that had been carefully

developed in order to create stable and consistent high turbidity conditions. However, both the turbidity and the *E. coli* concentrations of the feed water quality were adjusted to determine the filter response. The results indicate that percent reduction of both *E. coli* and turbidity will remain relatively constant in spite of drastic changes in sediment load and fecal contamination.

The target turbidity for phase 1, phase 2, and the beginning of phase 3 was 200 NTU. The actual turbidity measured averaged at 197 NTU with a standard deviation of 6.4. For the second half of phase 3, the target was increased to 1000 NTU. In this case, the actual averaged at 1065 NTU, with a standard deviation of 36. Comparing just filter 3c before and after the turbidity jump, the average turbidity reduction at 200 NTU was 50%, and the average at 1000 NTU was 57%. This amounts to a 450% increase in feed water turbidity, and a 360% increase in filtrate turbidity. In the face of such large fluctuations, a change in percent reduction from 50% to 57% is remarkably small. All filters exhibited similar behavior to filter 3c.

The target *E. coli* concentration in the feed water was 400 CFU / 100 mL; but specific tests range from below detection to 4000 CFU / 100 mL. While the percent reduction ranged significantly from one day to another, there was no correlation between the *E. coli* concentration and the percent reduction. The highest concentration measured had an average reduction of 52.3%, which was very typical of all the measurements made.

4.3.4 Presence of the Dual Sand Layer

One of the primary and foundational goals of this research project was to examine whether the addition of a Dual Sand Layer to the Biosand Filter is effective in overcoming the challenges associated with high turbidity source waters. If the modification is ineffective, optimizing it would be unnecessary. This section discusses both the need for modification to the BSF and the capacity of the DSL to meet that need based on all of the data collected, and especially on the targeted observations discussed so far.

The Necessity for a Modified BSF

The reason behind the general recommendation that the BSF not be used for high turbidity water is a combination of two assertions: high turbidity waters result in frequent maintenance; and

filter maintenance hinders treatment. If not for these two assertions, there would be no need to modify the BSF. However, under the conditions emulated in the laboratory, these two assertions were only marginally observed.

First, do high turbidity waters result in frequent maintenance? The laboratory feed water had turbidity of 200 NTU, with typical filtrate turbidity of approximately 100 NTU. One of the five laboratory filters had a typical maintenance frequency of about 20 operational days (Table 22), with 3 maintenance operations occurring over the course of the laboratory program. The control filter only required maintenance once, after 64 days of operation! The two cycles should be adjusted for two major differences between the two configurations: the head available to force water through and the surface area of the exposed sand. Accordingly, 1 to 1.5 months between maintenance operations would be expected for a household scale BSF with the laboratory feed water quality. This is much longer than the reported three days that some users have been trained to use in response to high turbidity conditions (Collin, 2009).

However, it is recognized that field conditions may vary significantly from the controlled laboratory conditions. Much higher turbidity feed water was observed to clog the filter more rapidly (Figure 22). Increases in the volume of water filtered each day had a similar effect (Figure 8). However, perhaps the most important potential change in the field conditions is to the water chemistry. In this experiment, the water chemistry was carefully controlled to result in highly stable suspended particles. This successfully emulated the reported field conditions that were used as a model. However, field studies report much higher turbidity reduction rates than those observed in the laboratory, indicating that the particles may be significantly less stable in those field conditions (Stevenson, 2008). This would greatly increase the clogging rate and the resulting maintenance frequency. The combination of these parameters may actually bring the maintenance frequency to a matter of days rather than months.

Comparing the laboratory data available here with reports regarding high turbidity field conditions, it appears that there are two categories of high turbidity water: high stability and low stability. High stability was modeled in the laboratory, but this does not result in high clogging

rates and frequent maintenance. Low stability appears to also exist, resulting in high turbidity reduction rates, high clogging, and frequent maintenance.

The next question is: does filter maintenance hinder treatment? Based on the laboratory observations, yes it does but only by about 10%. This is further discussed in Section 4.3.2.3. Maintenance does result in a short-term peak in turbidity, which can be avoided by flushing about 2 pore volumes through the filter. A long-term increase in turbidity is also observed, although only by approximately 12%. *E. coli* removal was measured as being slightly hindered by maintenance, although not in all cases, and only by an average of 10%. The day to day variability in *E. coli* removal rates is significantly higher than that.

In summary, the basic assertions underlying the need for a modified BSF were not observed to the same degree that was initially expected. There is a need for greater characterization of high turbidity field conditions, to determine the relative stability of suspended particles in different regions. It is suspected that the field conditions modeled in the laboratory are typical of high stability field conditions, but are not typical of the conditions which result in high clogging rates and frequent maintenance. At the same time, the laboratory results indicate that the decrease in treatment efficiency due to maintenance is measurable but is not as significant as variations in other parameters, such as the PV to filtrate ratio and the pause time. Nevertheless, if filter maintenance were to be necessary every three days, as is possible in very high turbidity yet low stability conditions, then pretreatment may be necessary. This would be particularly true if flushing 2 PV through the filter after maintenance were found to be a burdensome percentage of the treatable capacity of the filter between maintenance cycles.

The Benefits of the DSL

The next question to be examined is whether or not the DSL in particular is successful in alleviating the problems associated with high turbidity conditions. This will be separately addressed for high and low particle stability source water conditions, and summarized in the final paragraph.

For the conditions modeled in the laboratory, specifically high stability and high turbidity conditions, the DSL is found to be only marginally helpful in mitigating the impact of filter maintenance. Specifically, the DSL was not found to impact the long-term change in turbidity reduction or the impact on *E. coli* reduction. The only impact of the DSL on maintenance side effects was to decrease the sharpness of the short term peak in turbidity, for the first 2 PV. If this first 2 PV is a significant portion of the total maintenance cycle volume, users are unlikely to discard it in spite of training, and the DSL may be beneficial. If maintenance cycles are long enough that 2 PV can be flushed through and discarded without being a burden, then that would probably be a better option.

At the same time, the presence of the DSL was found to induce more frequent maintenance by two mechanisms: constricting the flow through a smaller area and therefore increasing the clogging rate; and lowering the available head. Only the first mechanism is expected in the full-scale design.

However, for high turbidity yet low stability conditions, the DSL may be significantly more beneficial. If the filter actually clogs so quickly that maintenance must be conducted every three days, then flushing 2 PV through the filter and discarding the filtrate would mean wasting 13% of the water that is collected by hand and carried to the home, assuming the filter is sized and operated according to the recommendations in Chapter 5. If it is determined that users will not discard of this water and would rather use it in spite of the turbidity peak, then adding a DSL would lessen the sharpness of the peak.

Another question to consider is whether the DSL has any impact on the long-term treatment efficiency or DO properties. In general, filter 1 (without a DSL) was right in the middle of all of the other filters for both *E. coli* and turbidity. No impact from the DSL was observed. The DO was definitely impacted by the addition of a DSL. However, the impact is much less important than the impact of pause time. Based on Figure 37, addition of 7 cm total thickness without changing any other parameters means that the DO levels will drop below 6 mg/L after 40 hours instead of after 50 hours. Fortunately, the filter is expected to be used multiple times every day, and this long pause time will rarely be a problem.

In summary, the DSL does not appear to be beneficial under all conditions. Under very high turbidity but low particle stability conditions, the DSL may decrease the turbidity by about 15% of an estimated 13% of the filtrate volume. It is expected that a more mature filter would only increase this benefit, perhaps eventually surpassing the unmodified filter in microbiological treatment, but this was not observed in the laboratory after one month of ripening and three months of testing. The additional complications in design, construction, operation and maintenance may outweigh the marginal benefit observed.

4.3.5 Applicability to Field Conditions

This research program was conducted in a relatively controlled laboratory setting, at the laboratory scale. One important question to address is how to scale the laboratory results to field conditions and field sizing. For the purpose of this analysis, calculations and design of the full-scale filter are taken from Clair Collin's field experiments (2009), which were conducted with a design for the BSF that is commonly used in Ghana.

The inner diameter of the main filter tube used in the laboratory was 10.16 cm. This is approximately 1/12 the cross-sectional area of the full-scale filter. The depth of the primary sand layer, coarse sand and gravel were all equal to the full-scale filter. Additionally, the same criteria were used for sieving and washing the sand and gravel media.

Accordingly, it is expected that flow volumes and pore volumes will scale linearly with area. Phase 1 was conducted with volumes of 2 L and 3 L, which corresponds to about 24 L and 36 L each day. This is a relatively small amount for a household, even in the developing world. Based on field observations by Matt Stevenson (2008), the actual volume required in a day is expected to be on the order of double that amount. This is expected to increase the clogging rates and maintenance frequencies observed by a factor of two.

While flow rate would theoretically scale up linearly within the sand bed itself, it is more sensitive to other aspects of the design. In particular, all of the head losses due to piping were effectively removed in the laboratory scale design, but using piping large enough to contribute

negligible head loss. In the plastic design used commonly in Ghana, significant head loss appears to occur when entering the pipe which collects water from the gravel base. In addition, the laboratory filters were built significantly taller than those used by Collin, to allow for room for experimentation. This increased the amount of water that could be poured in at a time, increasing the head pushing water through the filters. This was particularly true of the unmodified filter, and less true of filter 4c with the thickest DSL. This must be accounted for when scaling flow rates up to full-scale.

DO flux is expected to change at the full-scale. It is interesting to tie together a couple of observations. First of all, it was observed in Section 4.3.1.1 that it appears that most of the DO flux is occurring in the direct water channel around DSL. Assuming that the flux through the sand is negligible compared to the flux through the water, the addition of a DSL blocked 88% of the flux area. Comparing filter 1c with filter 4c, the length through which DO had to be transferred was increased from 5 cm to 12.3 cm, or about 2.5 times longer. Even still, in Figure 37 the DO of filter 4c was about 0.5 mg/L lower than filter 1c at all times. This resulted in filter 4c dropping below 6 mg/L after 40 hours, while filter 1c dropped below after 50 hours.

Temperature is expected to significantly change the filter operation, especially in respect to DO. The laboratory temperature was a relatively constant 23°C. At field conditions with higher temperatures, the DO solubility will decrease, and microbiological activity is expected to increase, resulting in higher DO demand, and potentially higher microbiological treatment efficiency.

Based on the above analysis, it appears that the DO drops slowly, and that the DO flux is not very important except at long pause times. If water is being filtered on a daily basis, then the DO will not be a major concern. Even accounting for lower DO solubility and higher oxygen demand, daily operation of the filter should provide plenty of oxygen to maintain aerobic conditions at the full-scale. If the DSL is selected for use, then the recommendations in Chapter 5 include considerations for maintaining a water channel between the *schmutzdecke* and the air surface.

5 Recommendations

This research considered the optimization of the Biosand Filter (BSF) for high turbidity waters, using a laboratory experimental procedure to test the impacts of adding a Dual Sand Layer (DSL) to the standard BSF design. The results relate to the DSL modification in particular as well as to the BSF design and operation in general.

This chapter is intended to apply the observations based on the laboratory data into specific useful conclusions. Recommendations are divided into three categories: specific recommendations for the DSL; general recommendations for any BSF; and questions for further research. The first two categories discuss both design and operation

5.1 The Use and Design of a Dual Sand Layer

5.1.1 When to Use the DSL

The DSL appears to be only potentially useful for conditions where high turbidity exists with low particle stability. Several simple tests can be conducted to determine whether the DSL may be applicable to a certain water supply.

- If unmodified BSF's are currently in use in the area, observing the average maintenance cycle would be the most direct method. If the filter absolutely must be maintained every 3 days or so in order to filter enough water for the household to use, then the addition of a DSL may improve the average quality of the filtrate.
- If no BSF's are in use, then a simple jar test can be used to estimate the clogging rate. Collect feed water in a jar or pot. Test the initial turbidity. Allow the water to settle for 24 hours, and retest the turbidity of water that is decanted off the top of the vessel. If the difference between the two is greater than about 300 NTU, then the addition of a DSL may improve the average quality of the filtrate.

For any other conditions, the DSL is not recommended. Addition of a DSL adds significant complication to the filter design, construction, operation, and maintenance. Under other conditions, The small benefit anticipated does not warrant this extra complication, since it is so

likely that complexity will result in ineffective use. Essentially the same benefit can be obtained by simply flushing the filter with 2 pore volumes after each maintenance operation.

Furthermore, even under the conditions where the DSL may be beneficial, using a large jar to collect feed water in and settle for 24 hours prior to filtration would probably be just as beneficial as the DSL.

5.1.2 How to Design the DSL

If the DSL is selected, the recommended design parameters are as follows:

- Diffuser:
 - Holes 2 mm in diameter
 - At least 1 hole per 25 cm²
 - Located so that all water pours through the DSL
- 5 cm thick supernatant water
- 5 cm thick fine sand
- 1 cm thick coarse sand
- Dual Sand Layer Stand
 - Holes 2 mm in diameter
 - At least 1 hole per 8 cm²
 - Designed to permit a 1 to 2 cm water channel reaching from the lower sand layer to the top of the top of the filter body, outside of the DSL stand but inside of the filter body.
 - Design to be removable, so that lower sand layer can be cleaned as necessary, and so that the DSL can be removed and set aside when periods of disuse for greater than 1 day are expected.
- Field testing of a full-scale, ripened DSL BSF at peak seasonal temperatures is recommended prior to distribution, in order to ensure that anaerobic conditions do not occur. Testing should consist of testing DO of the filtrate at several points before and after 75% of the pore volume has passed through. This should be conducted for several pause times, such as 12 hours, 24 hours, 48 hours, and 72 hours.

5.1.3 How to Use the DSL

Using the DSL requires a few additional tips beyond those required for an unmodified BSF. First, maintenance should always be conducted on the upper sand layer as the default. It is expected that the maintenance cycle will get shorter and shorter, until eventually maintenance of the upper sand layer does not restore the flow rate. At this point, the upper sand layer must be removed and the lower sand layer maintained. This should restore the flow rate.

Second, care must be taken to ensure that anaerobic conditions do not occur. With regular use following the above design recommendations, this should never happen. However, if travel or other circumstances cause the filter to be disused for more than a day, the DSL should be removed, and the water level should be decanted to about 5 cm deep above the lower sand layer.

5.2 The Use and Design of Any Biosand Water Filter

The best water to come out of any BSF especially under high turbidity conditions is water that was poured into the filter at last 24 hours earlier. However, even decreasing the residence time to 4 hours is a major improvement over water that was flushed through the filter without ever stopping. Accordingly, it is very important to know the volume of water that is retained within the grains of sand; the pore volume.

The BSF does not perform as a perfect plug flow reactor. That is to say that some mixing occurs, so that some of the water that was just poured into the filter will come out before the last drop of water that was in the filter during the pause time. Therefore, to ensure that all of the water that comes out of the filter has been in the filter for at least 4 hours, the most that should be poured into the filter at a time is 50% of the pore volume.

5.2.1 BSF Design

Based on the observations in Section 4.3, especially those regarding pause time, pore volume, and mixing, the following design parameters are recommended. The design recommendations are coordinated with the operation recommendations, in the following section. These recommendations are valid for any household scale BSF, and are intended to supplement the basic designs currently in use around the world. It is expected that these recommendations will

significantly improve the overall water quality of the filtrate. The degree of the improvement depends on the source water quality and the extent to which these recommendations are already being followed.

- Size the filter as big as practically possible and economically affordable. The target minimum size should be large enough that 200% of the pore volume is sufficient for all household uses.
- In sizing the filter, a balance of cross-sectional area and depth should be considered. Too much surface area without sufficient depth might hinder plug flow and facilitate mixing, so that some of the water which was just poured in will exit the filter at less than 50% of the pore volume. On the other hand, higher surface area means that clogging will occur less quickly and lower flow rates per unit area are tolerable, requiring less frequent maintenance.
- Size the freeboard such that 50% of the pore volume is the maximum amount of water that can be poured into the filter in a single pouring. This eliminates the need for the user to keep track of how much water is poured in at a time.
- The collection bucket should be sized to match 50% of the pore volume, so that users are not encouraged to top off the filter.
- The safe storage bin should be large enough that 100% or more of the pore volume can fit. The storage bin should not be the limiting factor in when filter operation can occur. If water is primarily used at certain times of the day, and water quality is enhanced if filtration occurs periodically, then sufficient storage must be available.
- Depending on the culture, it may be advisable to include some sort of timing device with the filter, such as an hourglass set to 4 hours, as a mechanism of keeping track of when the filter is available for use.

5.2.2 BSF Operation

Along with the design recommendations above, the following list of operational guidelines are recommended. These are good guidelines for all filter users, but are particularly important for users with high turbidity source waters. It is expected that these guidelines will result in the best possible water quality out of the filter. These recommendations will improve the water quality even if the BSF was not designed according to the design guidelines above. However, the design

guidelines above will make effective use easier and more likely to be followed, and will also ensure that sufficient water quantity is available while following these operational guidelines.

Daily Operation Guidelines:

- Pour 50% of the pore volume into the filter every time the filter is operated.
- Maintain a consistent operation schedule. Operate first thing in the morning, last thing in the evening, and at consistent intervals throughout the day.
- Operate the filter as few times as necessary to provide sufficient water quantity. If more water is necessary, operate the filter more frequently. Do not increase the operational water volume beyond 50% of the pore volume.
- Try to maintain at least 4 hours between filter operations, but not if this necessitates increasing the operational volume.
- Store water at home settling in a storage vessel as long as possible prior to filtration. This vessel should act as a pretreatment settling basin, and can be expected to remove as much as 50% of the turbidity in 24 hours. This will decrease the filter maintenance frequency.
- If the overnight pause time is the longest, use this water for drinking, especially for young children, sick or immunity deficient people, pregnant women, and the elderly.

Filter Maintenance Guidelines:

- Never maintain unless absolutely necessary due to low flow rate. In other words, when the next scheduled operation comes and the filter is still too full to fit the entire prescribed volume of feed water into the filter, perform filter maintenance.
- During filter maintenance, do not agitate the sand below 1 cm depth. Greater depths may inhibit the ability of the filter to provide microbiological treatment.
- Repeat the maintenance procedure thoroughly until continued stirring results in water of comparable turbidity with the feed water.
- Flush 2 pore volumes through the filter and discard the filtrate if possible. Filtrate may also be used for uses where water quality is less critical, or may be re-filtered.

5.2.3 Complementary Treatment Options

The BSF alone has significant potential for improving water quality at the household scale, regardless of the initial quality of the water supply. If only one HWTS technology is affordable, then the BSF is recommended for high turbidity waters. However, under high turbidity conditions the BSF does not remove all of the pathogens of concern. The average *E. coli* reduction rate measured under high turbidity conditions was 64%. Therefore, additional treatment options are highly recommended if affordable. If further treatment is not affordable, then it is better to drink the filtrate with only 36% of the initial pathogens rather than drink the source water with 100% of the initial pathogens.

If affordable, the first additional treatment option recommended is chlorination. Matt Stevenson (2008) provides a thorough description of two chlorination options, one using a liquid solution and another using solid tabs.

The recommended maximum turbidity of water to be chlorinated is 50 NTU. It is expected that the BSF, operated according to the guidelines listed above, will provide filtrate with turbidity less than 50 NTU, even under very high turbidity and high stability conditions. If conditions are such that all of the above recommendations are carefully followed as much as practical, and the filtrate is still above 50 NTU, then purchase of a second BSF is one potential solution. This will enable filters to be operated alternately, doubling the pause time, and increasing the turbidity reduction by several percent.

If slightly more treatment is affordable, then the next step recommended for high turbidity source waters is coagulation through *Moringa Oleifera* seeds (Lea, 2010). *Moringa Oleifera* trees grow in tropical climates, and the seeds can be ground up and used as a simple chemical coagulant for high turbidity waters. Michael Lea (2010) thoroughly describes the process of harvesting the seeds, testing the necessary dose in a village setting, and treating the water at a household scale. This would provide an excellent pretreatment option in many cases, and would greatly enhance the potential of the BSF to provide safe and plentiful drinking water in high turbidity conditions.

The combination of *Moringa Oleifera* seed coagulation, Biosand Filtration operated according to the above guidelines, and chlorination should turn almost all high turbidity fecally contaminated source waters into relatively clean and safe drinking water.

5.3 Further Study

This research included the experimental analyses of applying and optimizing Biosand Filters to high turbidity source waters. Progress was made, in particular with the development of a successful method for duplicating highly stable high turbidity conditions in the laboratory. These analyses present a number of opportunities for additional research.

For high turbidity waters with kaolinite as the primary clay type, raising the pH or increasing the ionic concentration has the potential to coagulate particles (see Section 2.4). This results in several related questions for further research:

- What are the impacts of pH and ionic concentration adjustment on other common types of clay, such as montmorillonite or illite?
- Is adjusting the pH of high turbidity water for the purpose of particle destabilization a feasible option?
- Is adjusting the ionic concentration of high turbidity water for the purpose of particle destabilization a feasible option? What ions can be added at low cost and maximum ionic concentration? What are the health impacts of the necessary concentrations?

It appears that high turbidity source waters can be categorized according to particle stability. More thorough categorization of the chemical and physical water quality properties of typical water supplies in developing countries would be highly beneficial to future research.

The combination of BSF with chlorination and *Moringa Oleifera* coagulation shows significant potential as a relatively low cost method of producing high quality water out of high turbidity source water. A feasibility study should be conducted, along with an economic analysis and disease burden reduction calculations.

Using the pore volume approach to intermittent slow sand filtration, as suggested here, is expected to result in significantly more rapid clogging at depth than is normal for SSF. While this provides good treatment of high turbidity waters, it may result in head losses that are not recovered by normal maintenance, resulting in the need for replacing sand media in as little as a few years. A long-term field study on the filter longevity implications of depth removal in high turbidity conditions is needed.

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Appendix A – Feed Water Target Criteria

In an unpublished internal report dated June 2007, Susan Murcott *et. al* documented the water quality and treatment properties of several Ghanaian water dugouts in both the dry and rainy seasons (Murcott, 2009). In the dry season, the turbidity ranged from 8 to 700 NTU, with an average of 250. The *E. coli* counts varied from 0 to 8,000 Colony Forming Units per 100 mL, with an average of 780 CFU per 100 mL. In the wet season, the turbidity ranged from 20 to over 2,000 NTU, with an average of 690 NTU. *E. coli* counts ranged from 0 to 1,700 CFU per 100 mL, with an average of 440 CFU per 100 mL.

In the above unpublished report, raw water from a Ghanaian dugout was observed as it settled for a period of one week. The turbidity began at 47 NTU. After 26 hours, the turbidity was 21 NTU; after 48 hours, 18 NTU; after 74 hours, 15 NTU; after 162 hours, 6 NTU. Sampling was always taken out of the top of the settling tank. After the week was over and the water was remixed, the turbidity returned to 30 NTU.

Many field studies have measured the effectiveness of a single sand layer BSF in treating turbidity and *E. coli*. For low turbidity waters, removing over 80% of the turbidity is common. Collin's control tests, with an average feed turbidity of 103 NTU, measured only 20% to 40% turbidity reductions, while still attaining 90% *E. coli* removal (2009). Kikkawa's field control tests, on the other hand, measured turbidity treatment efficiencies above 90% (2008). In 2008, Stevenson sourced several households using concrete biosand filters. He recorded typical influent turbidities of 200 to 500 NTU, and measured typical effluent turbidities less than 10 NTU. Based on the above data, in Ghana alone the BSF varies in turbidity treatment efficiency from 20% to 98%, with both extremes being documented for high turbidity water.

Based on the above descriptions of Ghanaian dugout water quality, and the purposes of this research, the target properties for the source water were developed, as described in Table 1.

Appendix B – Feed Water Data

Table 25 – Collection Log for Water and Wastewater

Date	Item	Volume (L)	Location
11/23/09	Water	40	Wachusett Reservoir - Location 1 ¹
12/1/09	Water	45	Wachusett Reservoir - Location 2 ²
12/3/09	Wastewater	0.25	UBWPAD ³
12/4/09	Water	45	Wachusett Reservoir - Location 2
12/8/09	Wastewater	1	UBWPAD
12/11/09	Water	47	Wachusett Reservoir - Location 2
12/22/09	Water	45	Wachusett Reservoir - Location 3 ⁴
1/7/10	Water	140	Wachusett Reservoir - Location 4 ⁵
1/20/10	Wastewater	1	UBWPAD
1/21/10	Water	135	Wachusett Reservoir - Location 4
2/5/10	Water	140	Wachusett Reservoir - Location 4
2/9/10	Wastewater	0.25	UBWPAD
2/22/10	Water	128	Wachusett Reservoir - Location 4
2/25/10	Wastewater	0.25	UBWPAD Note: heavy rainfall
3/8/10	Wastewater	0.25	UBWPAD
3/11/10	Water	140	Wachusett Reservoir - Location 4
3/17/10	Wastewater	0.25	UBWPAD
3/24/10	Wastewater	0.25	UBWPAD
3/27/10	Water	35	Wachusett Reservoir - Location 2
3/30/10	Wastewater	0.25	UBWPAD

Notes:

- 1) On the north side of the basin where the Quinepoxet River (and Quabbin Transfer Pipe) enter the reservoir. This basin is contained by the railroad on the east side of the basin. Sampling occurred at the shore, where the water was shallow and relatively high turbidity persisted.
- 2) From the railroad bridge where the basin from Location 1 enters the rest of the reservoir. Sampling occurred from the steps at the bridge abutment, where the water was well over 5' deep, with high flushing.
- 3) Upper Blackstone Water Pollution Abatement District. Collected from the primary clarifier effluent canal – prior to activated sludge injection.
- 4) Location 3 - Cosgrove Intake Structure, intake structure for the (Boston) MWRA Water Supply.
- 5) Location 4 - Clinton Pump Station, intake structure for the Town of Clinton water supply.

Table 26 – Ripening Phase Feed Water Manufacturing

Date	Volume (L)	Initial pH	NaOH ¹ (mL)	Adjusted pH	Kaolinite (g)	Wastewater (mL)
12/1/09 ²	11.0	6.30	7.00	8.30	11.010	50.0
12/2/09	11.0	6.40	5.00	8.06	1.501	50.0
12/3/09	11.0	6.70	6.00	8.04	1.500	50.0
12/4/09	11.0	6.67	6.00	8.09	2.002	50.0
12/7/09	10.8	6.75	6.00	8.24	2.250	50.0
12/8/09	11.0	6.61	6.00	7.95	2.250	50.0
12/9/09	11.0	6.50	6.00	7.95	2.250	50.0
12/10/09	11.0		7.00	7.85	2.250	50.0
12/11/09	11.0	6.45	6.00	8.39	2.250	50.0
12/14/09	11.0		6.00	8.49	2.250	50.0
12/15/09	11.0	6.47	5.00	8.20	2.250	50.0
12/16/09	11.0	6.45	6.00	8.10	2.250	50.0
12/22/09	5.5	6.69	2.67	8.80	1.125	25.0
12/28/09	5.5	6.69	3.16	7.70	1.125	25.0
12/30/09	5.5	6.69	3.16	7.70	1.125	25.0

Notes: 1) NaOH concentration was 0.1 N
2) Turbidity too high. Not poured through filters.

Table 27 – Ripening Phase Feed Water Quality

Date	Turbidity (NTU)	Final pH	E. Coli (CFU /100mL)
12/1/09 ¹	990	7.17	
12/2/09	95	7.24	
12/3/09	122	7.25	
12/4/09	172	7.15	
12/7/09	203	7.30	
12/8/09	182	7.12	
12/9/09	189	7.10	
12/10/09	167	7.00	
12/11/09	180	7.10	
12/14/09	194	7.00	
12/15/09	186	6.83	
12/16/09	189	7.15	
12/22/09	198	6.95	
12/28/09	186	6.85	
12/30/09	186	6.85	

Note: 1) Turbidity too high. Not poured through filters.

Table 28 – Phase 1 Feed Water Manufacturing

Date	Volume (L)	Initial pH	NaOH¹ (mL)	Adjusted pH	Kaolinite (g)	Wastewater (mL)
1/4/10	11	6.60	4.50	8.18	2.250	50.0
1/5/10	11	6.60	4.50	8.22	2.300	50.0
1/6/10	11	6.64	4.50	8.15	2.300	50.0
1/7/10	11	6.64	4.50	8.12	2.300	50.0
1/8/10	11		4.50	8.38	2.300	50.0
1/11/10	11	6.75	4.00	8.31	2.300	50.0
1/12/10	11	6.75	4.00	8.25	2.300	50.0
1/13/10	11	6.60	4.75	8.14	2.300	50.0
1/14/10	11	6.90	4.00	8.61	2.300	50.0
1/15/10	11	6.55	4.25	8.07	2.300	50.0
1/17/10	11	6.71	4.50	8.38	2.300	35.0
1/19/10	11	6.70	4.50	8.10	2.300	44.0
1/20/10	11	6.71	4.50	8.06	2.300	50.0
1/21/10	11	6.84	3.81	8.34	2.300	50.0
1/22/10	11	6.71	4.50	8.50	2.300	50.0
1/23/10	11	6.59	6.00	8.29	2.300	50.0
1/25/10	11	6.90	4.00	8.20	2.300	50.0
1/26/10	11	6.77	4.00	8.24	2.300	50.0
1/27/10	12	6.72	6.00	8.50	2.300	50.0
1/28/10	11.5	6.80	5.00	8.40	2.300	5.00
1/29/10	11.5	6.62	4.50	8.24	2.300	5.00
1/30/10	11.25	6.67	4.50	8.07	2.300	5.00
2/1/10	11.25	6.80	4.50	8.50	2.300	5.00
2/2/10	11.5	6.70	4.50	8.10	2.300	5.00
2/3/10	11.5	6.52	0.50	8.18	2.300	5.00
2/4/10	11.5		0.45		2.300	20.00
2/5/10	14	6.50	0.55	8.33	2.300	20.00
2/8/10	11.5	6.72	0.50	8.01	2.300	20.00
2/9/10	13	6.70	0.50	8.49	2.300	5.00
2/10/10	11.5	6.71	0.50	8.20	2.300	5.00
2/11/10	11.5	6.75	0.45	8.13	2.300	5.00
2/12/10	11.5	6.93	0.40	8.35	2.300	5.00
2/15/10	11.5	6.95	0.40	8.49	2.300	5.00
2/16/10	11.5	6.94	0.38	8.34	2.300	5.00
2/17/10	11.5	6.64	0.50	8.18	2.300	5.00
2/18/10	11.5	7.03	0.30	8.41	2.300	5.00
2/19/10	11.5	6.73	0.45	8.18	2.300	5.00
2/22/10	11.5	6.82	0.35	8.26	2.300	5.00
2/23/10	11.5	6.64	0.50	8.06	2.300	20.00

Note: 1) NaOH concentration was 0.1 N until 2/2/10. Beginning 2/3/10, concentration was 1.0 N.

Table 29 – Phase 1 Feed Water Quality

Date	Turbidity (NTU)	Final pH	E. Coli (CFU /100mL)
1/4/10	190	7.10	
1/5/10	201	7.05	
1/6/10	200	7.10	
1/7/10	200	7.05	
1/8/10	194	7.18	
1/11/10	201	7.25	
1/12/10	190	7.08	
1/13/10	188	7.18	
1/14/10	202	7.48	
1/15/10	196	7.16	
1/17/10	188	7.45	
1/19/10	190	7.26	
1/20/10	187	7.17	
1/21/10	200	7.26	
1/22/10	192	7.41	
1/23/10	200	7.40	
1/25/10	192	7.35	
1/26/10	196	7.25	4600
1/27/10	189	7.39	
1/28/10	196	7.64	620
1/29/10	199	7.45	
1/30/10	201	7.42	
2/1/10	203	7.64	
2/2/10	202	7.45	98
2/3/10	184	7.52	
2/4/10	193	7.34	172
2/5/10	199	7.49	
2/8/10	192	7.42	
2/9/10	206	7.54	700
2/10/10	194	7.22	
2/11/10	195	7.35	537
2/12/10	197	7.20	
2/15/10	182	7.53	
2/16/10	200	7.36	370
2/17/10	202	7.27	
2/18/10	200	7.22	260
2/19/10	195	7.22	
2/22/10	195	7.24	
2/23/10	202	7.33	35

Table 30 – Phase 2 Feed Water Manufacturing

Date	Volume (L)	Initial pH	NaOH ¹ (mL)	Adjusted pH	Kaolinite (g)	Wastewater (mL)
2/24/10	13.0	6.83	0.50	8.31	2.600	50.0
2/25/10	11.5	6.71	0.43	8.11	2.300	5.00
2/26/10	11.5	6.72	0.45	8.12	2.300	5.00
3/1/10	17.0	6.77	0.50	8.14	3.400	7.40
3/2/10	11.5	6.60	0.48	8.01	2.300	5.00
3/3/10	11.5	6.74	0.40	8.08	2.300	5.00
3/4/10	11.5	6.65	0.48	8.11	2.300	5.00
3/5/10	11.5	6.81	0.40	8.11	2.300	5.00
3/8/10	14.0	6.70	0.56	8.09	2.800	6.10
3/9/10	11.5	6.62	0.46	8.05	2.300	5.00
3/10/10	11.5	6.57	0.50	8.25	2.300	5.00
3/11/10	11.5	6.74	0.41	8.00	2.300	5.00
3/12/10	11.5	6.58	0.40	8.06	2.300	5.00
3/15/10	11.5	6.74	0.40	8.09	2.300	5.00
3/16/10	13.5	6.90	0.43	8.26	2.700	5.00
3/17/10	14.0	6.66	0.60	8.06	2.800	6.10
3/18/10	11.5	6.66	0.50	8.05	2.300	5.00
3/19/10	11.5	6.56	0.50	8.03	2.300	5.00
3/22/10	13.0	6.77	0.48	8.04	2.600	5.65

Note: 1) NaOH concentration was 1.0 N.

Table 31 – Phase 2 Feed Water Quality

Date	Turbidity (NTU)	Final pH	E. Coli (CFU /100mL)
2/24/10	215	7.10	
2/25/10	196		560
2/26/10	205	7.19	
3/1/10	194	7.31	
3/2/10	194	7.20	
3/3/10	202	7.15	
3/4/10	199	7.20	
3/5/10	193	7.17	
3/8/10	196	7.26	
3/9/10	200	7.17	
3/10/10	196	7.27	
3/11/10	207	7.21	
3/12/10	196	7.20	
3/15/10	212	7.20	
3/16/10	196	7.15	
3/17/10	195	7.31	367
3/18/10	199	7.25	
3/19/10	194	7.25	
3/22/10	207	7.25	

Table 32 – Phase 3 Feed Water Manufacturing

Date	Volume (L)	Initial pH	NaOH¹ (mL)	Adjusted pH	Kaolinite (g)	Wastewater (mL)
3/23/10	7	6.66	0.30	8.16	1.400	3.05
3/24/10	7	6.66	0.30	8.16	1.400	3.05
3/24/10	7	6.55	0.35		1.400	3.05
3/24/10	7	6.55	0.35		1.400	3.05
3/25/10	7	6.74	0.25	8.04	1.400	3.05
3/26/10	7.5	6.71	0.38	7.95	1.400	3.05
3/27/10	7	6.62	2.75	8.14	7.000	20.00
3/28/10	7	6.59	0.35	8.23	6.000	20.00
3/29/10	7	6.53	0.40	8.32	6.000	20.00
3/30/10	7	6.61	0.45	8.65	6.000	10.00
4/2/10	7	6.72	0.35	8.91	6.000	10.00
4/3/10	7	6.91	0.30	8.87	6.000	10.00
4/7/10						

Note: 1) NaOH concentration was 1.0 N.

Table 33 – Phase 3 Feed Water Quality

Date	Turbidity (NTU)	Final pH	E. Coli (CFU /100mL)
3/23/10	207	7.29	
3/24/10	207	7.29	
3/24/10	195	7.28	
3/24/10	195	7.28	
3/25/10	201	7.00	
3/26/10	185	7.11	76
3/27/10	1190	7.00	
3/28/10	1125	6.98	
3/29/10	1035	7.02	
3/30/10	1040	7.01	900
4/2/10	1058	7.21	
4/3/10	1063	7.30	
4/7/10			

Appendix C – Daily Filter Flow and Maintenance Data

This appendix presents the flow data collected for all five filters in each phase. For each data point in the ripening phase, phase 1, and phase 2, the start time was recorded, and subtracted from the time at which 1 Liter had passed through the filter. This was used to calculate the flow rate presented here. Phase 3 differs in that only 1 L was passed through filters 1, 2, 4, and 5. Accordingly, the time at which 0.5L had passed through these filters was recorded, and the flow rate calculated based on that.

Since the actual start and finish time only serve to calculate the flow rate, only the actual flow rates are presented, here. Note that the time between beginning the first filter and beginning the last filter generally ranged between 10 and 20 minutes.

Points marked with an asterisk (*) denote that filter maintenance occurred just prior to filter operation.

Table 34 – Ripening Phase Flow & Maintenance Data

Date	Flow Rate (Liters per minute)				
	1	2	3	4	5
12/2/09					
12/3/09					
12/4/09					
12/7/09	*	*	*	*	*
12/8/09					
12/9/09					
12/10/09					
12/11/09					
12/14/09					
12/15/09	0.231	0.231	0.226	0.197	0.222
12/16/09					
12/22/09					
12/28/09					
12/30/09					

Note: Flow data was only collected for one day during the ripening phase.

Table 35 – Phase 1 Flow Data

Date	Flow Rate (Liters per minute)				
	1	2	3	4	5
1/4/10	0.196	0.219	0.216	0.189	0.182
1/5/10	0.214	0.214	0.205	0.164	
1/6/10		0.213	0.204	0.197	0.152
1/7/10	0.199	0.203	0.201	0.174	0.141
1/8/10	0.194	0.200	0.190	0.163	0.123
1/11/10	0.206	0.210	0.202	0.176	0.126
1/12/10	0.199	0.205	0.197	0.164	0.113
1/13/10	0.197	0.215	0.192	0.156	0.098
1/14/10	0.193	0.207	0.185	0.163	0.085
1/15/10	0.179	0.197	0.180	0.144	0.085
1/17/10					
1/19/10	0.178	0.161	0.158	0.153	0.072
1/20/10	0.161	0.177	0.155	0.135	0.062
1/21/10	0.157	0.179	0.145	0.130	0.056
1/22/10	0.170	0.176	0.157	0.129	0.051
1/23/10	0.171	0.191	0.154	0.123	0.045
1/25/10	0.168	0.190	0.148	0.117	0.052
1/26/10	0.158	0.192	0.135	0.100	*0.166
1/27/10	0.167	0.181	0.137	*0.176	0.168
1/28/10	0.157	0.181	0.124	0.169	0.169
1/29/10	0.172	0.160	0.120	0.172	0.169
1/30/10	0.158	0.189	0.127	0.161	0.163
2/1/10	0.158	0.184	0.124	0.166	0.160
2/2/10	0.159	0.188	0.121	0.167	0.157
2/3/10	0.154	0.184	0.117	0.163	0.153
2/4/10	0.141	0.178	0.107	0.147	0.142
2/5/10	0.148	0.176	0.113	0.164	0.150
2/8/10	0.144	0.148	*0.164	0.159	0.137
2/9/10	0.143	0.178	0.163	0.152	0.120
2/10/10	0.144	0.171	0.167	0.153	0.128
2/11/10	0.137	0.167	0.170	0.146	0.122
2/12/10	0.138	0.157	0.164	0.146	0.114
2/15/10	0.135	0.169	0.166	0.149	0.123
2/16/10	0.135	0.171	0.161	0.133	0.102
2/17/10	0.127	0.162	0.159	0.141	*0.150
2/18/10	0.131	0.173	0.160	0.139	0.160
2/19/10	0.128	0.169	0.164	0.131	0.156
2/22/10	0.127	0.166	0.160	0.132	0.153
2/23/10	0.130	0.172	0.161	0.133	0.147

Table 36 – Phase 2 Flow Data

Date	Flow Rate (Liters per minute)				
	1	2	3	4	5
2/24/10	0.126	*	*	*	0.150
2/25/10	0.122	0.153	0.149	0.136	0.158
2/26/10			0.153	0.140	0.150
3/1/10	0.115	0.153	0.144	0.130	0.151
3/2/10	0.117	0.142	0.134	0.133	0.139
3/3/10	0.114	0.142	0.143	0.128	0.150
3/4/10	0.112	0.138	0.146	0.129	0.139
3/5/10	0.113	0.139	0.140	0.127	0.131
3/8/10	0.113	0.133	0.143	0.121	0.120
3/9/10	0.112	0.141	0.142	0.129	0.123
3/10/10	0.108	0.135	0.142	0.125	0.117
3/11/10	0.108	0.133	0.137	0.124	0.098
3/12/10	0.099	0.134	0.140	0.125	0.090
3/15/10	0.103	0.134	0.137	0.124	0.093
3/16/10	0.095	0.129	0.127	0.120	0.077
3/17/10	*0.150	0.132	0.125	0.117	*0.143
3/18/10	0.147	0.130	0.124	0.118	0.147
3/19/10	0.150	0.131	0.136	0.109	0.140
3/22/10		0.121	0.121		0.139

Table 37 – Phase 3 Flow Data

Date	Flow Rate (Liters per minute)				
	1	2	3	4	5
3/23/10	0.079	0.088	0.112	0.078	0.079
3/24/10	0.081	0.082	0.120	0.083	0.088
3/24/10	0.083	0.081	0.115	0.084	0.090
3/24/10	0.079	0.063	0.116	0.084	0.090
3/25/10	0.080	0.080	0.113	0.082	0.091
3/26/10	0.076	0.082	0.114	0.081	0.089
3/27/10	0.075	0.076	0.110	0.077	0.085
3/28/10	0.075	0.077	0.110	0.079	0.084
3/29/10	0.074	0.075	0.107	0.078	0.080
3/30/10					
4/2/10	0.071	0.069	0.088	0.076	0.072
4/3/10	0.069	0.073	0.084	0.071	0.070
4/7/10	0.068	0.067	0.078	0.076	0.064

Appendix D – Daily Filter Dissolved Oxygen and Pause Time Data

This appendix presents the dissolved oxygen data and the pause time calculations for phases 1, 2, and 3 of filter operation. Note that the time of operation was not recorded until January 13th, so pause time prior to this date is calculated assuming that the filter was run at 12 PM. In addition, the dissolved oxygen data prior to January 15th are deemed useful only for comparing the relative DO of each filter on any given day. By January 15th, the problems encountered had been resolved.

Table 38 – Phase 1 Dissolved Oxygen and Pause Time Data

Date	Time	Pause Time (hr)	Pause Time DO (mg/L)					0.75L DO (mg/L)					
			1	2	3	4	5	1	2	3	4	5	
1/4/10		120.0											
1/5/10		24.0											
1/6/10		24.0	9.1	9.2	9.1	9.1	9.1	9.5	10.0	9.7	9.9	10.0	
1/7/10		24.0	9.2	8.5	8.8	8.9	8.5	8.8	9.0	8.8	9.0	8.5	
1/8/10		24.0	9.1	8.4	8.8	8.4	8.3	7.9	8.0	8.1	7.9	7.9	
1/11/10		72.0	8.1	6.5	7.6	7.5	7.4	6.7	6.8	7.4	7.2	6.9	
1/12/10		24.0											
1/13/10	4:00 PM	28.0											
1/14/10	5:00 PM	25.0											
1/15/10	3:00 PM	22.0	7.80	6.37	7.36	7.13	7.48	7.57	7.16	7.38	7.37	7.90	
1/17/10	10:00 PM	55.0											
1/19/10	4:00 PM	42.0	7.86	6.44	6.68	6.50	7.27	7.06	6.44	6.54	6.69	6.81	
1/20/10	5:00 PM	25.0	8.07	6.66	6.92	6.75	7.48	8.33	7.22	7.43	7.16	7.31	
1/21/10	5:00 PM	24.0	8.27	7.19	7.18	7.07	7.87	7.93	7.59	7.56	7.79	7.91	
1/22/10	3:00 PM	22.0	7.98	6.45	7.13	6.90	7.79	7.73	7.27	7.39	7.23	7.72	
1/23/10	3:00 PM	24.0	7.73	6.30	7.02	6.80	7.70	7.01	6.89	7.23	6.91	7.37	
1/25/10	3:00 PM	48.0	7.12	5.08	6.30	5.86	7.21	6.21	5.54	5.96	5.48	6.35	
1/26/10	5:00 PM	26.0	7.52	5.07	6.40	6.12	7.41	7.03	7.00	7.01	6.34	6.90	
1/27/10	5:00 PM	24.0	8.02	5.90	6.95	6.60	7.66	6.73	6.59	6.93	7.28	7.00	
1/28/10	4:00 PM	23.0		6.11	7.15	6.68	7.55	7.78	7.06	7.26	7.32	7.61	
1/29/10	4:00 PM	24.0	7.97	6.26	6.86	6.75	7.39	6.88	7.40	7.02	7.02	6.92	
1/30/10	4:00 PM	24.0	7.84	6.41	7.08	6.83	7.30	7.25	6.59	6.91	6.69	6.83	
2/1/10	9:00 AM	41.0	7.65	6.56	7.11	6.75	7.21	6.99	6.02	6.69	6.39	6.42	
2/2/10	1:00 PM	28.0	8.08	7.09	7.26	7.05	7.46	7.52	7.05	7.05	6.84	6.76	
2/3/10	9:00 AM	20.0	7.44	6.99	7.31	6.90	7.22	7.64	7.37	7.20	7.26	6.95	
2/4/10	10:00 AM	25.0	8.27	7.28	7.72	7.37	7.58	7.86	7.39	7.58	7.75	7.48	
2/5/10	4:00 PM	30.0	8.31	7.23	7.80	7.62	7.85	8.08	7.03	7.63	7.40	7.57	
2/8/10	5:00 PM	73.0	7.59	6.50	7.03	7.45	7.38	5.85	5.37	5.85	5.96	6.08	
2/9/10	1:00 PM	20.0	8.26	7.07	7.17	7.03	7.36	7.57	7.04	7.55	7.49	7.41	
2/10/10	4:00 PM	27.0	7.85	6.89	7.16	7.23	7.46	7.33	6.80	7.07	7.30	7.04	
2/11/10	5:00 PM	25.0	7.98	7.04	7.31	7.22	7.69	7.75	7.22	7.85	7.20	7.17	
2/12/10	3:00 PM	22.0	8.16	7.17	7.46	7.31	7.56	7.81	7.25	7.33	7.47	7.26	
2/15/10	5:00 PM	74.0	8.05	6.32	6.67	6.27	7.01	6.47	5.04	6.57	6.54	5.85	
2/16/10	2:00 PM	21.0	8.05	6.94	7.00	6.83	7.19	7.62	7.30	7.43	7.38	7.02	
2/17/10	3:00 PM	25.0	7.85	6.90	7.23	6.98	7.45	7.79	7.55	7.30	7.56	7.83	
2/18/10	4:00 PM	25.0	7.68	6.66	7.19	6.90	7.09	7.17	6.57	6.92	7.05	6.64	
2/19/10	3:00 PM	23.0	7.62	6.68	7.10	6.78	7.18	7.21	7.07	7.39	7.15	7.10	
2/22/10	9:00 AM	66.0	8.09	6.07	6.67	6.40	7.06	6.55	5.79	6.30	5.85	6.23	
2/23/10	2:00 PM	29.0	8.24	7.01	7.14	6.90	7.08	7.52	7.05	7.41	7.15	7.09	

Table 39 – Phase 2 Dissolved Oxygen and Pause Time Data

Date	Time	Pause Time (hr)	Pause Time DO (mg/L)					0.75L DO (mg/L)				
			1	2	3	4	5	1	2	3	4	5
2/24/10	6:00 PM	28.0	7.77				7.16	7.03				6.39
2/25/10	4:00 PM	22.0	7.30	6.61	6.50	6.01	6.95	7.07	4.67	4.62	4.09	6.27
2/26/10	3:00 PM	23.0	7.36	5.93	6.16	5.46	6.79	6.95		4.64	4.60	6.57
3/1/10	6:00 PM	75.0	7.91		6.28	5.47	6.74	6.08	3.49	4.29	2.68	5.18
3/2/10	3:00 PM	21.0	8.17	7.70	6.87	6.43	7.18	7.32	6.26	6.10	6.14	7.07
3/3/10	5:00 PM	26.0	7.87	7.55	7.20	6.73	7.05	7.16	6.19	5.95	5.63	6.57
3/4/10	5:00 PM	24.0	7.84	7.63	7.37	6.96	7.25	7.37	6.34	6.54	6.39	6.41
3/5/10	4:00 PM	23.0	8.13	7.66	7.42	6.93	7.10	7.47	6.55	6.31	6.29	6.73
3/8/10	4:00 PM	72.0	7.89	7.15	6.58	6.15	7.02	5.37	4.50	4.92	4.26	5.08
3/9/10	3:00 PM	23.0	7.69	6.86	6.96	6.46	7.00	7.07	6.13	6.48	6.31	6.71
3/10/10	11:00 PM	32.0	8.15	7.39	7.23	7.00	7.54	6.94	5.83	5.95	6.08	6.55
3/11/10	6:00 PM	19.0	8.25	7.14	7.21	6.82	7.50	7.54	6.67	7.19	6.95	7.30
3/12/10	11:00 PM	29.0	8.43	7.65	7.56	7.57	7.88	7.65	6.50	6.90	6.72	7.17
3/15/10	6:00 PM	67.0	8.01	7.27	6.92	6.57	7.11	6.15	4.96	5.20	4.68	5.59
3/16/10	9:00 PM	27.0	8.28	7.15	7.20	6.82	7.29	7.16	6.49	6.44	6.49	6.83
3/17/10	7:00 PM	22.0		7.36	7.45	7.35		7.59	7.07			
3/18/10	5:00 PM	22.0	8.10	7.27	7.33	7.26	7.32	7.40	6.88	6.83	6.75	7.19
3/19/10	7:00 PM	26.0	8.07	7.31	7.42	7.19	7.28	7.21	6.77	6.57	6.55	6.84
3/22/10	6:00 PM	71.0	8.17	7.10	7.13	6.48	6.95	5.11	4.39	4.88	4.65	5.77

Table 40 – Phase 3 Dissolved Oxygen and Pause Time Data

Date	Time	Pause Time (hr)	Pause Time DO (mg/L)					0.75L DO (mg/L)				
			1	2	3	4	5	1	2	3	4	5
3/23/10	12:30 PM	18.5	8.18	6.78	7.16	7.09	7.16	7.35	7.06	6.90	7.05	7.17
3/24/10	10:30 AM	22.0	8.41	7.46	7.76	7.70	7.39	7.41	7.31	7.06	7.22	7.26
3/24/10	2:30 PM	4.0	8.31	7.37	7.81	7.61	7.41	8.01	7.94	7.92	7.81	7.80
3/24/10	10:30 PM	8.0	8.29	7.55	7.77	7.77	7.63	7.85	7.74	7.52	7.64	7.65
3/25/10	11:00 AM	12.5	8.24	7.49	7.77	7.82	7.78	7.62	7.52	7.21	7.45	7.40
3/26/10	2:00 PM	27.0	8.00	7.52	7.54	7.45	7.38	7.06	6.88	6.25	6.89	6.75
3/27/10	6:00 PM	28.0	8.35	7.77	7.54	7.54	7.63	7.06	6.95	6.52	6.79	6.82
3/28/10	10:00 PM	28.0	7.73	7.54	7.33	7.22	7.40	6.79	6.97	6.16	6.67	6.69
3/29/10	5:00 PM	19.0	7.74	7.34	7.41	7.33	7.26	6.82	6.98	6.32	6.75	6.72
3/30/10	4:00 PM	23.0	7.41	7.26	7.06	7.12	7.13	6.42	6.56		6.29	6.27
4/2/10	4:00 PM	72.0	6.70	6.98	6.31	6.58	6.66	4.92	4.74	3.49	4.58	4.68
4/3/10	5:00 PM	25.0	7.81	6.98	6.67	6.75	6.95	6.60	6.45	6.08	6.68	6.30
4/7/10	4:00 PM	95.0	6.00	6.79	6.19	6.24	6.33	4.63	4.12	2.96	4.18	4.33

Appendix E – Daily Filter Treatment Data

In this appendix, the treatment efficiency of the BSF's is presented in terms of turbidity and *E. coli*. In the turbidity tables, data points with an asterisk (*) signify that filter maintenance occurred prior to filter operation. Turbidity samples were taken in 30 mL vials beginning at the designated point, (e.g., 1.50 L to 1.53 L). *E. coli* samples were approximately 120 mL. Unless otherwise noted, in phases 1 and 2, *E. coli* samples were taken immediately after the day's turbidity sample. Phase 3 *E. coli* samples were taken after 0.50 L had passed through the filter.

Table 41 – Ripening Phase Turbidity Values Across Filters

Date	Turbidity (NTU)					
	Timing:	1.5 L	1.5 L	1.5 L	1.5 L	1.5 L
	Feed Water	Filter 1	Filter 2	Filter 3	Filter 4	Filter 5
12/2/09	95.0					
12/3/09	122					
12/4/09	172					
12/7/09	203	*	*	*	*	*
12/8/09	182					
12/9/09	189					
12/10/09	167					
12/11/09	180					
12/14/09	194					
12/15/09	186	80.0	90.2	83.5	92.1	59.2
12/16/09	189					
12/22/09	198					
12/28/09	186					
12/30/09	186					

Table 42 – Phase 1 *E. coli* Concentrations Across Filters

Date	E. Coli (CFU / 100mL)					
	Feed Water	Filter 1	Filter 2	Filter 3	Filter 4	Filter 5
1/26/10	4600	2540	2490	1900	2310	1740
1/28/10	620	220	247	240	340	300
2/2/10	98	30	50	29	26	36
2/4/10	165	4	13	13	4	6
2/9/10	700	320	395	420	413	280
2/11/10	525	260	353	380	300	260
2/16/10	345	152	210	220	153	156
2/18/10	260	60	84	82	64	78
2/23/10	30	4	4	3	4	1

Note: *E. coli* concentrations below 40 CFU / 100 mL are approximate.

Table 43 – Phase 1 Turbidity Values Across Filters

Date	Turbidity (NTU)					
	Timing:	1.5 L	1.5 L	1.5 L	1.5 L	1.5 L
	Feed Water	Filter 1	Filter 2	Filter 3	Filter 4	Filter 5
1/4/10	190	80.0	90.2	83.5	92.1	59.2
1/5/10	201	91.7	96.2	86.7	92.1	61.0
1/6/10	200	95.7	103	96.2	105	66.3
1/7/10	200	91.0	98.4	91.4	98.1	60.0
1/8/10	194	94.2	101	92.5	101	59.7
1/11/10	201	91.1	103	94.2	98.8	58.9
1/12/10	190	95.4	103	94.7	98.5	57.9
1/13/10	188	90.7	100	89.5	93.4	50.6
1/14/10	202	93.9	99.4	88.7	91.8	43.3
1/15/10	196	97.0	103	92.5	91.5	48.1
1/17/10	188					
1/19/10	190	94.5	102	92.1	94.9	49.8
1/20/10	187	102	112	98.3	101.0	53.7
1/21/10	200	97.0	108	96.1	98.8	48.8
1/22/10	192	101	111	97.8	96.2	53.4
1/23/10	200	101	109	96.3	97.2	49.4
1/25/10	192	103	111	101	93.5	57.5
1/26/10	196	104	112	101	93.5	*160
1/27/10	189	106	115	103	*130	105
1/28/10	196	98.1	104	92.0	101	97.0
1/29/10	199	104	111	97.6	107	109
1/30/10	201	111	116	106	113	110
2/1/10	203	104	113	106	106	105
2/2/10	202	108	117	99.5	106	106
2/3/10	184	96.9	101	88.9	97.4	96.2
2/4/10	193	100	110	96.2	104	102
2/5/10	199	110	122	109	114	111
2/8/10	192	102	108	*137	103	103
2/9/10	206	104	116	109	107	97.4
2/10/10	194	97.1	105	101	98.4	91.6
2/11/10	195	101	112	108	103	95.5
2/12/10	197	95.0	105	102	99.5	89.5
2/15/10	182	92.3	103	101	99.0	92.8
2/16/10	200	97.4	112	105	105	93.0
2/17/10	202	91.8	102	101	97.9	*122
2/18/10	200	98.1	111	106	105	108
2/19/10	195	97.7	110	106	103	105
2/22/10	195	92.8	109	101	97.4	102
2/23/10	202	102	114	111	104	106

Table 44 – Phase 2 *E. coli* Concentrations Across Filters

Date	E. Coli (CFU / 100mL)					
	Feed Water	Filter 1	Filter 2	Filter 3	Filter 4	Filter 5
2/25/10	560	169	110	92	66	176
3/2/10	227	100	102	102	94	112
3/4/10	74	16	10	13	10	16
3/9/10	580	247	340	220	267	293
3/17/10	367	157	135	144	146	158
3/22/10	1	6 ¹			3 ¹	

Note: *E. coli* concentrations below 40 CFU / 100 mL are approximate.

- 1) Tested after 0.5 L had passed through filter. Therefore, initial *E. coli* correspond to the previous day's feed water. Apparently at this point the decay rate in the feed water stock was greater than it had been, previously.

Table 45 – Phase 2 Turbidity Values Across Filters

Date	Turbidity (NTU)					
	Timing:	1.50 L	1.75 L	1.75 L	1.75 L	1.75 L
	Feed Water	Filter 1	Filter 2	Filter 3	Filter 4	Filter 5
2/24/10	215	106	*	*	*	112
2/25/10	196	96.0	75.4	63.4	58.7	102
2/26/10	205	99.1		85.6	74.4	109
3/1/10	194	94.6	89.0	87.5	80.3	104
3/2/10	194	99.2	94.4	92.8	87.3	102
3/3/10	202	100	96.0	94.5	94.5	102
3/4/10	199	96.9	100	99.3	94.8	104
3/5/10	193	95.2	97.1	96.5	92.0	98.8
3/8/10	196	92.8	96.2	93.7	92.8	98.5
3/9/10	200	94.2	97.6	96.1	92.5	95.4
3/10/10	196	90.4	91.7	92.7	89.8	91.6
3/11/10	207	94.2	96.7	96.9	93.5	94.5
3/12/10	196	90.7	95.3	98.0	94.5	86.3
3/15/10	212	97.3	103	103	104	93.3
3/16/10	196	95.2	98.8	101	97.5	87.9
3/17/10	195	*143	94.5	95.2	95.3	*125
3/18/10	199	109	101	99.1	101	107
3/19/10	194	102	92.4	92.5	88.3	103
3/22/10	207	107	104	100	101	112

Table 46 – Phase 3 *E. coli* Concentrations Across Filters

Date	E. Coli (CFU / 100mL)					
	Feed Water	Filter 1	Filter 2	Filter 3	Filter 4	Filter 5
3/26/10	76	50		36 ¹	14	
3/30/10	900	7	1	1 ²	0	3
4/3/10	1	4	0	7 ³	4	6

Note: *E. coli* concentrations below 40 CFU / 100 mL are approximate.

- 1) Tested after 1.78L had passed through filter. (All other values in phase 3 tested after 0.50 L had passed through filter. Therefore, initial *E. coli* concentrations actually correspond to the previous day's feed water.
- 2) Also tested filter 3 at 1.78 L: 267 CFU / 100 mL
- 3) Also tested filter 3 at 1.78 L: 1 CFU / 100 mL

Table 47 – Phase 3 Turbidity Values Across Filters

Date	Turbidity (NTU)					
	Timing:	0.70 L	0.70 L	1.75 L	0.70 L	0.70 L
	Feed Water	Filter 1	Filter 2	Filter 3	Filter 4	Filter 5
3/23/10	207	69.2	29.9	110	31.8	73.3
3/24/10	207	66.5	26.8	97.3	29.9	50.8
3/24/10	195	75.7	43.1	93.2	47.3	63.8
3/24/10	195	70.5	33.0	94.0	38.2	54.6
3/25/10	201	72.1	33.8	100	38.1	56.8
3/26/10	185	71.6	29.2	91.2	22.5	51.2
3/27/10	1190	140	31.9	455	49.3	74.1
3/28/10	1125	179	42.7	509	64.2	108
3/29/10	1035	203	63.6	453	82.6	138
3/30/10	1040	196	60.9	445	82.2	136
4/2/10	1058	179	47.7	439	64.9	101
4/3/10	1063	226	78.6	427 ¹	96.5	141
4/7/10	1105	176	47.7	417 ²	59.7	104

Note:

- 1) Also tested turbidity of filter 3 after 0.70 L had passed through the filter: 138 NTU.
- 2) Also tested turbidity after 0.70 L: 76.6 NTU, and again from 1.78 L to 1.90 L: 434 NTU.