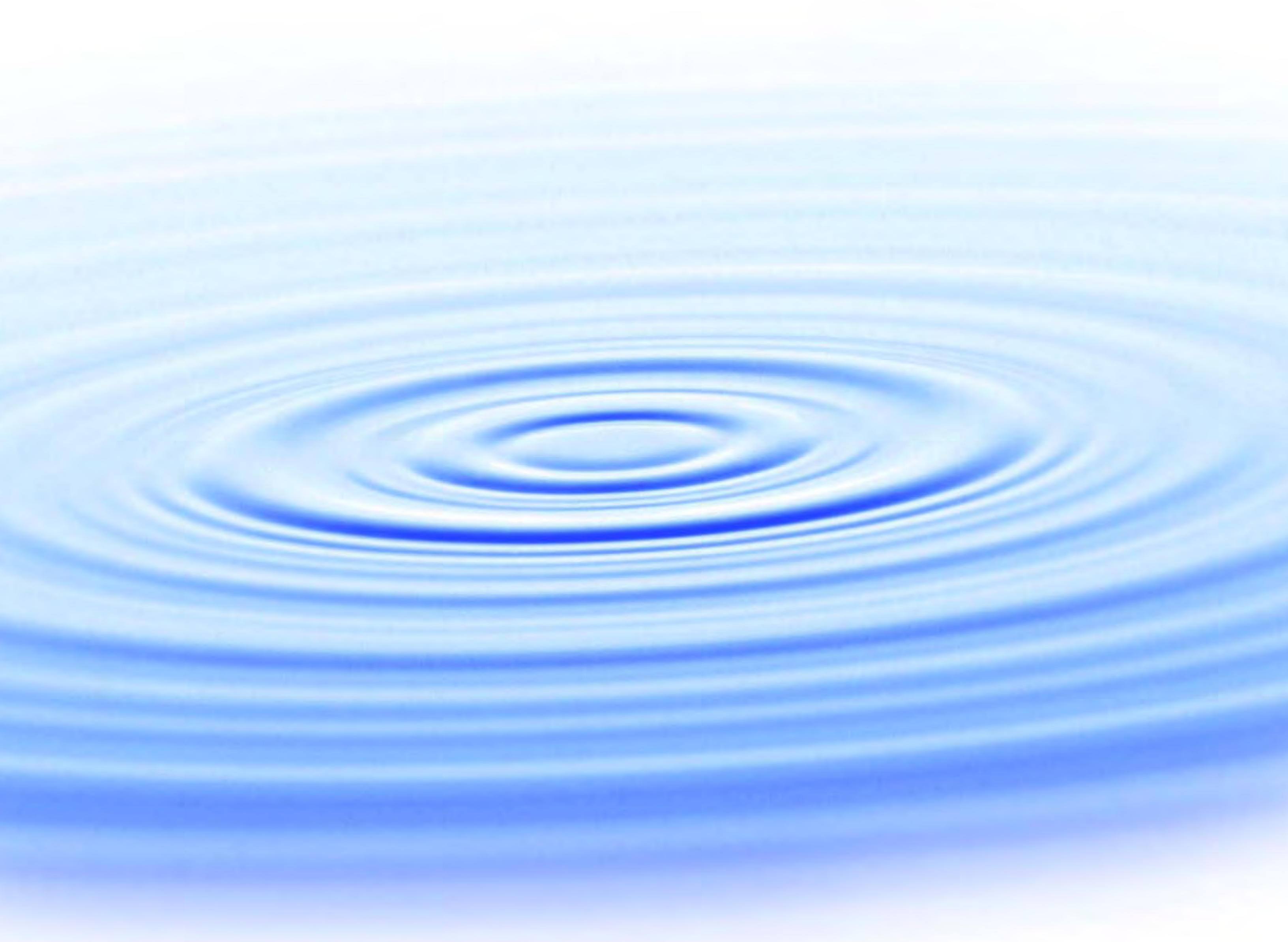
Filter Loading Evaluation for Water Reuse



WateReuse Research Foundation

Filter Loading Evaluation for Water Reuse

About the WateReuse Research Foundation

The mission of the WateReuse Research Foundation is to conduct and promote applied research on the reclamation, recycling, reuse, and desalination of water. The Foundation's research advances the science of water reuse and supports communities across the United States and abroad in their efforts to create new sources of high quality water through reclamation, recycling, reuse, and desalination while protecting public health and the environment.

The Foundation sponsors research on all aspects of water reuse, including emerging chemical contaminants, microbiological agents, treatment technologies, salinity management and desalination, public perception and acceptance, economics, and marketing. The Foundation's research informs the public of the safety of reclaimed water and provides water professionals with the tools and knowledge to meet their commitment of increasing reliability and quality.

The Foundation's funding partners include the Bureau of Reclamation, the California State Water Resources Control Board, the California Energy Commission, and the California Department of Water Resources. Funding is also provided by the Foundation's subscribers, water and wastewater agencies, and other interested organizations.

Filter Loading Evaluation for Water Reuse

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ABBREVIATIONS

ACH	aluminum chlorohydrate
ASCE	American Society of Civil Engineers
AWWA	American Water Works Association
CBHL	clean bed head loss
CCI	construction cost index
CCRWQCB	Central Coast Regional Water Quality Control Board
CDPH	California Department of Public Health
DCU	Digital Communication Unit
DDSD	Delta Diablo Sanitation District
EPA	Environmental Protection Agency
FLEWR	Filter Loading Evaluation for Water Reuse
HRT	hydraulic residence time
ISP	intensive sampling period
LACSD	Sanitation Districts of Los Angeles County
MGD	million gallons per day
MPN	most probable number
MRWPCA	Monterey Regional Water Pollution Control Agency
NTU	nephelometric turbidity units
NWRI	National Water Research Institute
O&M	operating and maintenance
OPS	Operations Staff
PVS	Pomona Virus Study
RO	reverse osmosis
RWQCB	Regional Water Quality Control Board
SCM	streaming current monitor
SJSC	San Jose/Santa Clara
SRO	single rate optimization
UCB	University of California at Berkeley
USAID	U.S. Agency for International Development
WPCP	Water Pollution Control Plant
WRF	WateReuse Research Foundation

FOREWORD

The WateReuse Research Foundation, a nonprofit corporation, sponsors research that advances the science of water reclamation, recycling, reuse, and desalination. The Foundation funds projects that meet the water reuse and desalination research needs of water and wastewater agencies and the public. The goal of the Foundation's research is to ensure that water reuse and desalination projects provide high-quality water, protect public health, and improve the environment.

An Operating Plan guides the Foundation's research program. Under the plan, a research agenda of high-priority topics is maintained. The agenda is developed in cooperation with the water reuse and desalination communities including water professionals, academics, and Foundation subscribers. The Foundation's research focuses on a broad range of water reuse research topics including:

- Defining and addressing emerging contaminants
- Public perceptions of the benefits and risks of water reuse
- Management practices related to indirect potable reuse
- Groundwater recharge and aquifer storage and recovery
- Evaluation and methods for managing salinity and desalination
- Economics and marketing of water reuse

The Operating Plan outlines the role of the Foundation's Research Advisory Committee (RAC), Project Advisory Committees (PACs), and Foundation staff. The RAC sets priorities, recommends projects for funding, and provides advice and recommendations on the Foundation's research agenda and other related efforts. PACs are convened for each project and provide technical review and oversight. The Foundation's RAC and PACs consist of experts in their fields and provide the Foundation with an independent review, which ensures the credibility of the Foundation's research results. The Foundation's Project Managers facilitate the efforts of the RAC and PACs and provide overall management of projects.

The Foundation's primary funding partners include the Bureau of Reclamation, California State Water Resources Control Board, the California Energy Commission, Foundation subscribers, water and wastewater agencies, and other interested organizations. The Foundation leverages its financial and intellectual capital through these partnerships and other funding relationships.

The overall goal of the Filter Loading Evaluation for Water Reuse project was to address scientific, engineering, and regulatory gaps related to the impact of filter loading rate on granular media, rapid depth filtration of wastewater. Higher filter loading rates would allow more water to be recycled with minimal cost implications. Because of the regulatory implications of the project, the California Department of Public Health was consulted on a regular basis and was directly involved in establishing equivalency criteria for filter effluent quality.

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This study would not have been possible without the insights, efforts, and dedication of many individuals and organizations. These include the members of the research team and PAC members (as identified blow); the WateReuse Research Foundation's project manager, Josh Dickinson; and many key individuals at the participating utilities and related organizations. In addition, NWRI provided invaluable guidance throughout the project, via the Research Advisory Board under the leadership of Dr. Ron Linsky and Jeff Mosher, as well as the NWRI Technical Advisory Committee, whose members included Dr. Takashi Asano, Dr. Bob Carnahan, Jack De Marco, Dr. Harvey Collins, Dr. Mike Kavanaugh, and Dr. George Tchobanoglous. The research team would like to thank the WateReuse Research Foundation for funding this applied research project, as well as the following organizations for their in-kind contributions: Dublin San Ramon Services District, West and Central Basin Municipal Water Districts, City of San José, City of Santa Rosa, and Sanitation Districts of Los Angeles County.

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EXECUTIVE SUMMARY

The overall goal of the Filter Loading Evaluation for Water Reuse (FLEWR) project was to address scientific, engineering, and regulatory gaps related to the impact of filter loading rate on granular media, rapid depth filtration of wastewater. In particular, the current California Water Recycling Criteria restrict the filter loading rate to less than or equal to 5 gal/ft^2•min. Higher filter loading rates would allow more water to be recycled with minimal cost implications. Because of the regulatory implications of the project, the California Department of Public Health (CDPH) was consulted on a regular basis and was directly involved in establishing equivalency criteria for filter effluent quality.

The FLEWR project was conducted in two phases. During Phase I, the impact of filter loading rate (5, 6.25, 7.5, 8.75, 10 gal/ft^2-min; 12.2, 15.3, 18.3, 21.4, 24.4 m/h) on filter performance and effluent quality was investigated at a pilot-scale filtration facility. The results from the pilot plant provided evidence to obtain regulatory approval to test loading rates of 5.0 and 7.5 gal/ft^2-min (12.2 and 18.3 m/h) at several full-scale facilities during Phase II of the project. Additional pilot-scale experiments were also conducted during Phase II to provide a better understanding of the impact of coagulation on virus removal during filtration.

The pilot plant was constructed at the Monterey Regional Water Pollution Control Agency (MRWPCA) water recycling facility in Marina, CA. Secondary effluent seeded with MS2 coliphage served as influent to the pilot plant. Pretreatment at the pilot plant consisted of coagulation (proprietary blend of aluminum chlorohydrate and cationic polymer) followed by three flocculation chambers. The coagulated wastewater was fed to five identical filter columns (1 ft sand, 4 ft anthracite) that could operate at different loading rates. Online instruments collected influent and effluent data on turbidity and particle counts (2–15 μm diameter), and head loss data (using pressure transducers installed at various depths in the columns). Grab samples were analyzed for total coliform bacteria, *E. coli*, and MS2 coliphage. Batch disinfection of filter effluent samples was performed with sodium hypochlorite.

More than 200 pilot filter runs were conducted during four test periods in Phase I. During the first three test periods, filters loaded at 5, 6.25, 7.5, 8.75, and 10 gal/ft^2-min received the same influent water (the coagulant dose was the same for all loading rates). The conclusions from these test periods are as follows:

- When the coagulant dose was the same for all loading rates, the removal efficiency of the filters decreased for all metrics (turbidity, particle counts in 2-15 μm size range, total coliform bacteria, *E. coli*, and MS2 coliphage) as the loading rate increased.

- Although not explicitly investigated, it was observed that when filters performed well (i.e., high particle removal), the disparity between loading rates was greater. Likewise, when filters had low particle removal, the differences in filter performance between loading rates was minimal.

- Consistent with clean bed filtration theory, larger particles were removed more efficiently over the size range of 2 to 15 μm.

- The impact of loading rate on particle removal efficiency was consistent with filtration theory. However, the impact of loading rate was lower during clean bed treatment than later in the filter run.

- The apparent removal of MS2 coliphage by filtration alone was small (0.1 to 1 log) and was greater when higher coagulant and polymer doses were used. The apparent removal by coagulation plus filtration was much greater and increased with chemical dose (up to 3-log removal).

- Based on the head loss profiles in the filter bed, particles penetrated further in the filter bed at higher filter loading rates.

- Minimal particle removal occurred through the sand layer (1 ft; 31 cm) compared to the anthracite layer (4 ft; 122 cm).

- The filter loading rate did not have a subsequent impact on the ability to disinfect the effluent with chlorine, even when higher loading rates had significantly lower particle removal.

During the fourth test period in Phase I, only two loading rates were tested: 5.0 and 7.5 gal/ft^2-min. The two loading rates were tested on an alternating basis (all five filters tested at the same rate and switching the rate tested between runs) such that the coagulant dose could be continuously optimized for each loading rate to produce an effluent turbidity of 1.9 NTU (Nephelometric Turbidity Units; a similar turbidity target to the full-scale plant). The conclusions from this test period are as follows:

- Under these conditions, equivalent[1] filter effluent quality was produced at 7.5 gal/ft^2-min compared to 5.0 gal/ft^2-min with respect to turbidity, particle counts, and removal of indicator bacteria.

- The average coagulant dose necessary to achieve equivalent performance was 62% higher at the 7.5 gal/ft^2-min rate (5.6 mg/L of coagulant, versus 3.5 mg/L at 5.0 gal/ft^2-min).

- The higher coagulant dose resulted in significantly higher removal of MS2 coliphage at the higher loading rate (1.58 versus 0.25 log removal for the 7.5 and 5.0 gal/ft^2-min rates, respectively). The majority of the removal at 7.5 gal/ft^2-min was attributed to the coagulation step (e.g., aggregation).

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[1] A set of criteria was developed for demonstrating equivalent performance at a higher filter loading rate, compared to performance at 5.0 gal/ft^2-min, which was defined as standard practice. The criteria were established with input from the California Department of Public Health and the Technical Advisory Committee for the FLEWR project. The criteria were (1) no increase in average effluent turbidity; (2) no increase in effluent particle counts in the size ranges of 2–5 and 5–15 µm; (3) no decrease in log removal of MS2 phage; and, (4) no decrease in log inactivation of total coliform bacteria through subsequent disinfection.

- At the top of the filter bed, particle removal was highest at the 5.0 gal/ft^2-min rate, but deeper in the media bed (below 6 in from top) particle removal was higher at the 7.5 gal/ft^2-min rate.

- The filter loading rate did not have a subsequent impact on the ability to disinfect the effluent with chlorine.

During Phase II, additional experiments were conducted at the pilot facility to better understand the role of coagulation and particle association on virus removal. Three different types of bacteriophage (MS2, PRD1, and ΦX174) were used in these experiments. Key conclusions are:

- Coagulation was necessary to achieve effective removal of phage by filtration.

- With coagulation, greater than 2-log removal of MS2 and PRD1 was observed, whereas insignificant removal of ΦX174 was observed. These differences are likely due to differences in the surface characteristics of the viruses.

- Viruses removed by filtration were primarily associated with particles in the 0.4 to 12 μm size range.

During Phase II, full-scale experiments were conducted at two facilities. Sixty-two filter runs were completed at the MRWPCA facility and 40 runs were completed at the City of San Jose. Both facilities have dual media (sand and anthracite) filters. The removal efficiencies of turbidity and particles (2–15 μm) were assessed with online instruments, and ability to disinfect filter effluents was assessed with batch tests in the laboratory. At both facilities, equivalent effluent quality was produced at the two filter loading rates, as determined by criteria defined by California Department of Public Health[2].

Additional conclusions are as follows:

- To achieve equivalent performance at MRWPCA, the average coagulant dose was 51% higher when operating at 7.5 than when operating at 5.0 gal/ft^2-min (7.7 versus 5.1 mg/L, respectively). At San Jose, equivalent water quality was produced at both loading rates without the addition of a coagulant prior to filtration.

- At MRWPCA, the impact of loading rate on the removal efficiency was different for the three particle removal metrics. To produce equivalent effluent particle counts in

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[2] The equivalency criteria for Phase II of the project are:

1. No significant* increase in mean turbidity of filter effluent;
2. No significant* increase in mean concentration of 2–5 and 5–15 μm particles in filter effluent; and
3. No significant decrease in the ability to disinfect filter effluent

*Where significant increase = $\dfrac{0.2\ \text{NTU}}{\text{NTU produced at 5.0 gal/ft}^2\text{-min}}$ (reported as percent).

the 5–15 μm size range at both rates, the coagulant dose had to be optimized such that the 2–5 μm particle counts and turbidity were actually lower at 7.5 gal/ft^2-min than at 5.0 gal/ft^2-min.

- At San Jose, the increases in turbidity, 2–5 μm particles, and 5–15 μm particles were 5%, 3.1%, and 11%, respectively, when the filter loading rate was 7.5 gal/ft^2-min compared to 5.0 gal/ft^2-min. None of these increases were statistically significant.

A key overall finding from the project was that the negative impact of increased filter loading rate on treatment performance was more apparent when effective coagulation was practiced prior to filtration. Thus, the impact of loading rate was greater when the removal efficiency of the filters was higher. At San Jose, it was not necessary to use coagulant to meet the 2 NTU standard because the influent wastewater (secondary effluent) already had low turbidity. In contrast, to achieve the required removals to meet the 2 NTU standard in Monterey, significant coagulant doses were necessary. The resulting impact of loading rate was minimal at San Jose, where no coagulant was used at either loading rate; but Monterey was required to use 51% higher coagulant dose to produce equivalent effluent quality at the higher loading rate.

The observed relationship between turbidity and particle counts was complex. The ratio of turbidity to particle counts in the secondary effluents was different at San Jose and Monterey. Furthermore, turbidity and particles were removed with different efficiencies from each other and were different at the two treatment facilities. As a result, the particle counts in the San Jose filter effluents were higher than the Monterey filter effluents, despite the lower turbidity at San Jose. Thus, a turbidity requirement of 2 NTU is not likely to result in similar particle counts and size distributions in filter effluents from different water recycling plants.

Most specific to this study, however, was the observation that the impact of loading rate on removal efficiency was not consistent for turbidity and particles. At the pilot plant, the decrease in removal efficiency of turbidity and particles as the loading rate increased was similar. However, at both full-scale facilities, as the loading rate increased, the decrease in removal of larger particles (5–15 μm) was greater than for smaller (2–5 μm) particles and turbidity.

Clearly, coagulation and flocculation may influence the relationship between turbidity and particles, as these processes alter both the numbers of particles, as well as the particle size distribution. Unfortunately, the on-line instruments did not allow a complete characterization of the particle size distribution, as only particles in the 2–15 μm size range were measured. Given that the turbidity measurement is most sensitive to particles in the 0.1 to 1 μm size range, it is not surprising that the different particle metrics were not always correlated.

Despite its inability to mimic particle counts, turbidity is still recommended as the regulatory parameter for filter effluent quality. Particle counts are not recommended for several reasons:

- Online instruments only measure a small segment of the particle size distribution (2–15 μm).

- Online instruments are not currently reliable, as they cannot handle high particle counts present in some wastewaters, the data are highly variable, biological growth in the instrument tubing causes clogging, and accurate calibrations can be difficult.

- There is insufficient information to establish acceptable effluent particle counts that are protective of public health.

Using the results from the Phase II full-scale testing at the Monterey facility, MRWPCA requested and received a permanent waiver to operate their tertiary filters at a loading rate of 7.5 gal/ft^2-min. As a result, MRWPCA can provide tertiary treatment for its entire design flow until the year 2030 (projected). The cost savings were estimated to be $5.5 million for the period 2009 through 2030, as a result of not having to build new filters as well as a slight savings in operation and maintenance. The potential savings if other facilities in California are allowed to operate tertiary filters at higher loading rates could be very large, but will vary depending on the unique circumstances at each treatment plant.

Several areas requiring further research were identified in this study, including investigating other ways to increase treatment plant capacity without capital improvements, developing a broader scientific base for other water recycling regulations, and improving our understanding of the role of particle-association in pathogen removal. In addition, the following specific research areas are recommended:

- Filter loading rates higher than 18.3 m/h, both with and without precoagulation, could be tested at facilities that have good quality secondary effluent (e.g., San Jose).

- Chloramines are less effective at inactivating viruses than free chlorine, but the California Water Recycling Criteria do not distinguish between free and combined chlorine. Research is needed to validate metrics and procedures that ensure complete nitrification and free-chlorine disinfection (e.g., online testing for free chlorine and ammonia). If such a study were successful in persuading CDPH to give credit for free chlorine disinfection, chlorine contact basins could be designed with much smaller footprints, significantly reducing the cost of recycled water.

- The potential for coagulation to improve virus removal at treatment plants that do not currently coagulate prior to filtration should be characterized.

- Development of more comprehensive filter performance metrics is needed.

- The current California Water Recycling Criteria require 5-log reduction of virus to be demonstrated for alternative tertiary treatment processes, using seeded MS2 coliphage or poliovirus. This requirement should be revisited in light of the lower reductions achieved by currently approved processes (such as filtration followed by chloramination).

- Additional mechanistic research is needed on the impact of coagulation (different coagulants and polymers, and range of doses) on virus removal, especially on actual human pathogenic viruses. Further experiments comparing the ability to disinfect the filter effluent are also recommended to determine if the increase in virus-particle association that improved virus filterability would result in particle shielding during disinfection. This research approach could also be extended to protozoan cysts.

- Indigenous phage and enteric viruses may behave differently than seeded phage. With the advent of quantitative molecular detection methods for nonculturable organisms, removal and mechanisms of pathogens native to the wastewater is possible. As detection limits and costs decrease, these molecular techniques can be applied to better characterize removal mechanisms of indigenous organisms through treatment processes.

CHAPTER 1

INTRODUCTION

Granular media, rapid depth filtration is a widespread tertiary treatment process used to prepare wastewater for recycling. The filter loading rate (flow rate normalized by filter surface area) is one of the most important parameters impacting the performance of filters. The FLEWR project was initiated to address key knowledge gaps with respect to the impact of loading rate on wastewater filtration. The project specifically addressed loading rate in the context of California's existing Water Reuse Regulations. California is the only state to specify a maximum filtration rate for reclaimed water, which currently is set at 5 gallons per minute per square foot (12 m/h). The capacity of many California water reclamation facilities is constrained by the regulatory filtration rate limit. A better understanding of the impact of loading rate on tertiary filtration would inform the development of improved guidelines, which could have large implications on the cost of water recycling. The background for the FLEWR project, the specific research objectives, and the project team are described in the following sections of this chapter.

1.1 CALIFORNIA WATER REUSE REGULATIONS

In the United States, water reclamation is regulated on the state level; there are no federal regulations for water reclamation. Of the 50 U.S. states, 26 have water reuse regulations 15 provide water reuse guidelines, and 9 have neither regulations nor guidelines (U.S. Environmental Protection Agency (EPA) and U.S. Agency for International Development [USAID] 2004). California has the strictest regulations of any state, and as a result, many water reclamation facilities through the United States and the world look to California for guidance and strive to meet their regulations.

The California "Water Recycling Criteria" encompass all the State regulations governing production, distribution, and use of recycled water for 43 specifically approved applications. These regulations are also commonly referred to as "Title 22" (Cal. Code Regs., 2006), even though a portion of the Water Recycling Criteria are encoded in Title 17 of the California administrative code and even though Title 22 covers a range of other subjects besides recycled water.

Of particular significance to the subject of filter loading rates is Section 60301.320 in Title 22 (titled "Filtered wastewater"), which reads (emphasis is added to illustrate the pertinent words):

"Filtered wastewater" means an oxidized wastewater that meets the criteria in subsection (a) or (b):

> (a) Has been coagulated and passed through natural undisturbed soils or a bed of filter media pursuant to the following:
>
> > (1) At a rate that *does not exceed 5 gallons per minute per square foot of surface area* in mono, dual or mixed media gravity, upflow or pressure filtration systems, or does not exceed 2 gallons per minute per square foot of surface area in traveling bridge automatic backwash filters; and

(2) So that the turbidity of the filtered wastewater does not exceed any of the following:

(A) An average of 2 NTU within a 24-hour period.
(B) 5 NTU more than 5 percent of the time within a 24-hour period.
(C) 10 NTU at any time.

(b) Has been passed through a microfiltration, ultrafiltration, nanofiltration, or reverse osmosis membrane so that the turbidity of the filtered wastewater does not exceed any of the following:

(1) 0.2 NTU more than 5 percent of the time within a 24-hour period; and

(2) 0.5 NTU at any time.

1.2 WATER RECYCLING IN MONTEREY COUNTY

Seawater intrusion into the aquifers underlying the northern Monterey County farmlands has progressed steadily toward Salinas over the last 50 years as a direct result of over-pumping for irrigation and municipal water supply. After about 20 years of planning and preliminary preparations, the Monterey County Water Recycling Projects were completed with the financial and institutional collaboration of numerous local, regional, state, and federal agencies. The Monterey Regional Water Pollution Control Agency has been delivering disinfected tertiary recycled water for irrigation of 12,000 acres of vegetables in Northern Monterey County since April 1998.

Title 22 Section 60301.320 is now the basis of design for all tertiary-treated recycled water production facilities in California, limiting the filter loading rate to 5 gal/ft^2-min. However, in 1997 when the Monterey Regional Water Pollution Agency's regional treatment plant was being upgraded to produce tertiary recycled water for irrigation of food crops, this limit on filter loading rate was not a requirement. As a result, the filtration system was designed on the basis of a loading rate of 7.5 gal/ft^2-min. Subsequently, this filter loading rate became a part of the conditions underlying the revolving loan funding provided by the State Water Resources Control Board to MRWPCA.

Thus, the stage was set for a conflict between meeting the Agency's grant condition and meeting regulatory limits on the operation of tertiary filters. This conflicting situation became even more critical when farmers' demand for recycled water during the peak irrigation seasons of July and August outstripped the capability of the treatment plant to filter water at an adequate rate. Unable to meet all the demand with recycled water, the shortfall is closed, every summer, with additional pumping of the coastal aquifers—already overdrafted and subject to seawater intrusion. Obviously, this is an undesirable situation, because the recycled water irrigation project was intended to reduce overdraft of the aquifer and slow or stop seawater intrusion in the first place.

1.3 REVIEW OF FILTER LOADING RATE LITERATURE

Filter loading rate affects the production capacity and treatment ability of a filter. The effect of loading rate on filtration has been studied empirically via lab and full-scale experimental studies, as reviewed here in sections 1.3.1 and 1.3.2. Although filtration practice remains highly empirical, advancements in filtration theory provide conceptual understandings of the factors affecting the filtration process, as reviewed in section 1.3.4. Few investigators have attempted to bridge the gaps between filtration theory and practice, and the ability of filtration theory to explain the impact of loading rate at an actual treatment facility had not been tested prior to this project.

1.3.1 Granular Media Filtration of Wastewater

In the 1960s and 1970s rapid sand filtration gained popularity as a particle removal step for tertiary wastewater treatment, especially in the context of water reuse (Culp, 1963; Tchobanoglous and Eliassen, 1970). Before this time, wastewater filtration was not commonly practiced, as filter runs were often too short to make the process economically feasible. In the 1960s, dual-media filtration gained popularity as a replacement for monomedia wastewater filtration. In dual-media filtration the wastewater first passes through a coarser media (e.g., anthracite) and then through a finer media (silica sand), which reduces the degree of surface clogging and extends the length of filter runs. This move led to longer and more economical filter runs and subsequently granular-media filtration found widespread application as a tertiary wastewater treatment process.

By 1974, the California health department required that tertiary treatment for reuse consist of coagulation, flocculation, sedimentation, filtration, and disinfection. Because only limited data existed on virus removal through tertiary treatment, the state was hesitant to approve any alternative technologies or treatment configurations (LACSD, 1977). This need for more data led to the “Pomona Virus Study” (PVS) conducted by the Sanitation Districts of Los Angeles County (LACSD) from 1975 to 1976. One objective of the PVS was to provide the necessary data that the health department would need to make informed regulatory decisions regarding alternative treatment trains. Subsequently, the PVS had great influence on the current California Water Recycling Criteria. For example, the PVS set the precedent that water recycling facilities in California be able to demonstrate a 5-log (99.999%) reduction in polio virus through tertiary treatment. Another important outcome of the study was that a maximum filter loading rate of 5 gal/ft^2-min was adopted as an additional safety parameter. It is important to note that the PVS investigated four different treatment trains, three of which used dual-media rapid-depth filtration (the fourth looked at carbon filters) and the loading rate was the same (5 gal/ft^2-min) in all three scenarios. Thus, the State of California adopted the maximum loading rate limit mainly because sufficient data at higher rates did not exist. It is likely that the 5 gal/ft^2-min rate used during the PVS was seen as the practical upper limit on loading rate (i.e., higher rates meant shorter and thus more costly runs). Thus, further work to study the impact of increased loading rate on tertiary filter performance was needed for regulators to better determine loading rate limitations.

Several previous studies have examined the effect of loading rate on filter performance (see Table 1.1). In some of these studies decreased filter efficiency was observed at higher loading rates (Adin and Elimelech, 1989; Baumann and Huang, 1974; Darby et al., 1991; FitzPatrick and Swanson, 1980; Tchobanoglous, 1970; Tchobanoglous and Eliassen, 1970); whereas in others only minimal effects were observed (Bench et al., 1981; Dawda et al., 1978; Tebbutt, 1971). However, none of these studies adequately separated the treatment performance attributed to coagulation versus filtration, nor sufficiently studied the effect of coagulation pretreatment on the impact of filtration rate.

1.3.2 Loading Rate History in Drinking Water Treatment

Several studies have been conducted to determine the impact of loading rate on drinking water filter performance. From the introduction of rapid filtration in the 1880s until the 1950s, 4.9 m/h was considered the practical upper limit of filtration loading rate operations. From 1950 to 1970, several rapid granular filtration experiments were performed to empirically determine if adequate treatment could be achieved at loading rates higher than 4.9 m/h (Table 1.1). Although it was found in these early studies that loading rates as high at 24.4 m/h could produce acceptable drinking water, these studies were empirically based and limited by the metrics used to evaluate filter performance. Further, these studies were highly specific to the influent water source and treatment plant of each study. The majority of this research was performed prior to the development of the filtration models used today. As a result, pilot-scale filtration studies are generally required for treatment plants to operate at these increased loading rates. Currently, drinking water rapid granular filtration processes are typically designed for a loading rate of 4.9 to 17.1 m/h (American Water Works Association [AWWA] and American Society of Civil Engineers [ASCE] 1998).

1.3.3 Filtration Theory

Several theoretical filtration models have been developed to understand particle removal through granular filtration. Particle removal during filtration is typically described as a two-step process. In the first step, particles are transported from the bulk flow to the filter media collector, and in the second step, particles attach to the collector surface or to other previously attached particles. The simplest filtration models are clean bed models, which predict particle removal prior to filter ripening and only consider particle attachment to the collector surface. These clean bed models provide an understanding of particle removal mechanisms, but there exists a large gap between theory and the performance of actual filtration systems. As a result, these models are not considered predictive and are not used in filter design. Transient state models have been developed that attempt to model postripening filter performance. However, these models are of limited use for understanding the impact of loading rate because they are highly empirical and require multiple fitted parameters. Nonetheless, their complexity emphasizes the fact that many dynamic factors may influence the impact that loading rate has on filtration performance at actual treatment plants.

Filtration models are typically validated with experimental data generated from laboratory column studies using ideal spherical particles and collectors. Such idealized experiments have demonstrated good correlation with model predictions (Tufenkji and Elimelech, 2004). However, no comparisons have been made between how changes in loading rate predicted by models compare with filter performance under actual field conditions, representative of treatment. Clean bed removal models predict that particle removal efficiency decreases with an increased loading rate because it (a) decreases the time for particle transport to the media collector, and (b) increases fluid shear and hydrodynamic forces, decreasing the particle attachment efficiency.

Other implications of increased loading rate include (a) changes in filtration rate may effect the particle size distribution of water as it will promote more in situ flocculation that is due to increased Reynolds number, as increased shear forces break particles apart, and (b), increased loading rate has been shown to result in deeper particle penetration, reducing head loss at the top of the media bed, and increasing the treatment capacity before reaching terminal head loss.

Table 1.1. Summary of Literature on Filter Loading Rate in Drinking Water and Wastewater

Reference	Water	Scale	Rates (gal/ft^2-min)	Pretreatment	Media (d_{10} in mm)	Metric(s)	Main Loading Rate Finding
Fuller (1898)	DW	Full	≤ 2	Coagulation	proprietary		No conclusion reached, the upper limit on loading rate not found
Baylis (1956)	DW	Full	2.0, 4.0, 4.5, 5.0	Conventional		TC, BC CPF	No effect on TC/BC, but higher CPF weight with higher loading rate
Cleasby and Baumann (1962)	DW	Pilot	0.7-9	Direct versus Conventional	sand (0.5)	TB	Performance is more dependent on pretreatment
Tchobanoglous and Eliassen (1970)	WW	Pilot	2, 5.8, 10	--	sand (0.49, 0.68, and 0.98)	SS/TB	Removal decreased with increasing loading rate
Westerhoff (1971)	DW	Full	5-10	Conventional	sand (0.45) vs. mixed (1.0, 0.45, 0.15)	TB, COL, Me, MC	Higher bacterial counts at rates greater than 6 gal/ft^2-min
Tebbutt (1971)	WW	Pilot	1.7-8.5	--	sand (0.5-1) and anthracite (1.0-2.5)	SS, BOD, COD	No or minimal effect
Baumann and Huang (1974)	WW	Pilot	2-7	--	sand and anthracite of various sizes	TB and SS	Decrease in performance with increased loading rate, especially at start of run
Hutchison (1976)	DW	Pilot	2.4, 4.8, 7.2	Direct	sand (0.45) and anthracite (n.s.)	TB	Higher rates are more susceptible to raw water problems.
Dawda et al. (1978)	WW	Pilot	3, 4, 5, 6	--	anthracite (1.1) and sand (0.45)	BOD, SS, VSS	No or minimal effect
FitzPatrick and Swanson (1980)	WW	Full	.8, 4.4	--	anthracite (1.4-2.7) and sand (0.6-1.4)	SS and others	Decrease in performance with increasing loading rate
Bench et al. (1981)	WW	Pilot	2-5	--	4 different bed configurations	BOD and SS	No or minimal effect
Adin and Elimelech (1989)	WW	Pilot	3.2, 6.5	--	sand (0.7)	SS and PC	Decreased particle removal with increased loading rate
Darby, Lawler, Wilshusen (1991)	WW	Lab	2.7-5.4	--	glass spheres (1.4-1.7 and 1.7-2.0)	SS	Slight decrease performance comparing 2.7 and 6.5 gal/ft^2-min
Rajala et al. (2003)	WW	Lab	2, 4.1		sand (0.9-1.2)	SS, TB, FC, CP, SRC	More variability at higher rates
Dugan and Williams (2004)	DW	Pilot	2, 4	Contact	anthracite (1.0) and sand (0.44)	TB, PC, CR	Decreased performance

Table 1.1. Continued

Abbreviations used in Table 1.1	
BC = Bacterial Count	MC = Microscopic Count
BOD = Biochemical Oxygen Demand	Me = Metal-Ion concentration
COD = Chemical Oxygen Demand	PC = Particle Counts
COL = Color	SRC= Sulphite Reducing Clostridia
CP= Coliphage	SS = Suspended solids
CPF = Cotton Plug Filter	TB = Turbidity
CR= Cryptosporidium	TC = Total Coliform Bacteria
DW = Drinking water	VSS = Volatile Suspended Solids
FC= Fecal Coliform Bacteria	WW = Wastewater

1.4 PROJECT OBJECTIVES

The FLEWR project was divided into two phases of activities. The goals of Phase I were to

1. Investigate the impact of filter loading rate (5, 6.25, 7.5, 8.75, 10 gal/ft^2-min; 12.2, 15.3, 18.3, 21.4, 24.4 m/h) on filter performance and effluent quality at the pilot scale.

2. Characterize filter performance and effluent quality sufficiently to seek approval from regulatory agencies to operate full-scale tertiary filters higher than 5 gal/ft^2-min during Phase II (as determined by the California Department of Public Health Equivalency criteria).

The goals of Phase II were to

1. Investigate the impact of filter loading rate (5 and 7.5 gal/ft^2·min; 12.2 and 18.3 m/h) at the following five full-scale treatment plants:
    a. Monterey Regional Water Pollution Control Agency
    b. City of San Jose Water Pollution Control Plant
    c. Delta Diablo Sanitation District
    d. City of Santa Rosa Subregional Water Reuse System
    e. Sanitation Districts of Los Angeles County

2. Conduct additional laboratory and pilot-scale experiments on virus removal mechanisms during filtration.

Because of the regulatory implications of the project, the California Department of Public Health (CDPH) was consulted on a regular basis throughout the project and was directly involved in establishing equivalency criteria for filter effluent quality.

1.5 PROJECT TEAM

The FLEWR project was accomplished by a large team, as shown in Figure 1.1. Dr. Bahman Sheikh was the lead Principal Investigator and was responsible for overall project planning, administration, presentations, and reporting. Dr. Kara Nelson, Associate Professor in Civil and Environmental Engineering at U.C. Berkeley, was responsible for designing the research plan, analyzing the data and making presentations. Gordon Williams, a Ph.D. student at U.C. Berkeley, was the lead researcher throughout the project. Bob Holden, the Water Recycling Coordinator at MRWPCA, was responsible for financial management and coordinating experiments that took place at the MRWPCA treatment facility during Phases I and II. Gordon and Bob designed and managed construction of the pilot filtration plant. Tom Kouretas, an engineer at MRWPCA, was responsible for running the pilot filtration plant. Dr. Bob Cooper, President of BioVir Laboratories, advised the project on microbiological analysis. Dr. Jim Crook was the liaison for interactions with the CDPH.

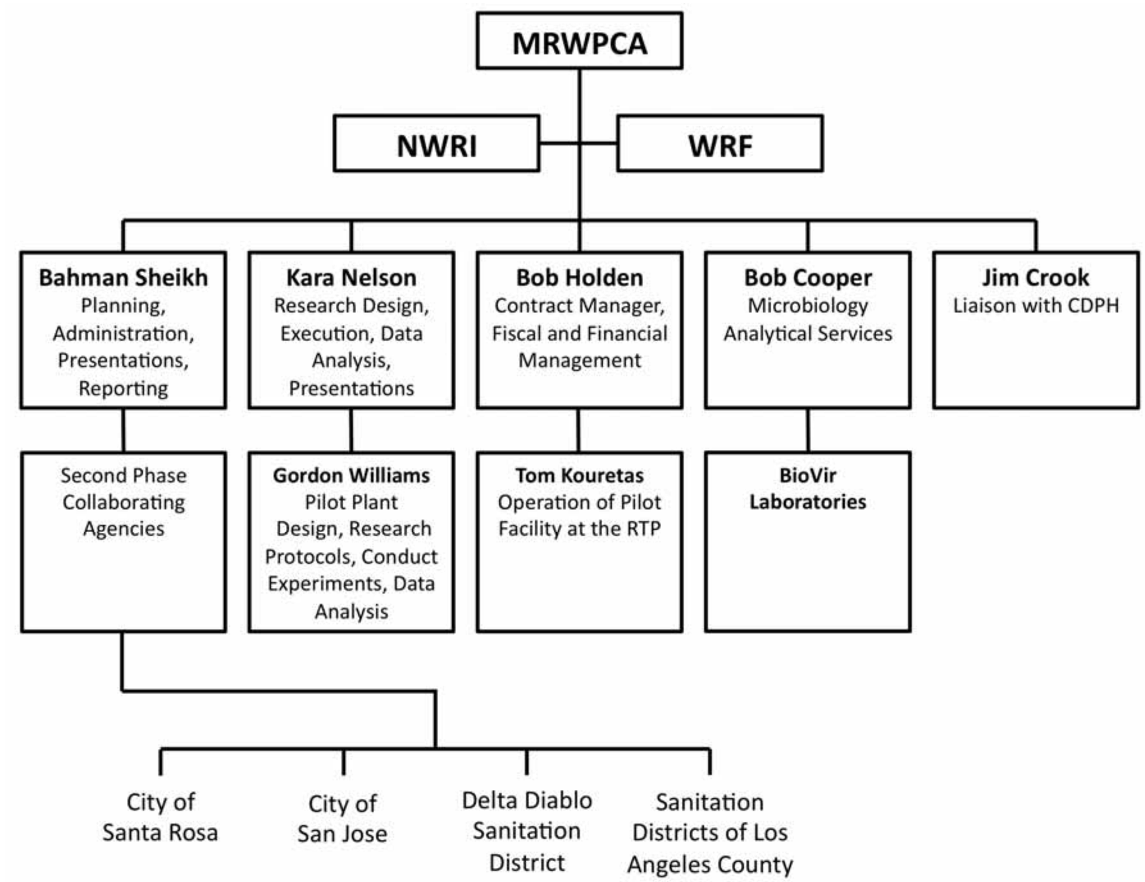


Figure 1.1. FLEWR project team and responsibilities.

CHAPTER 2

OVERVIEW OF PHASE I ACTIVITIES

Phase I of the FLEWR project consisted of a series of experiments conducted at a pilot filtration facility constructed at the MRWPCA water recycling plant in Marina, CA. In this chapter, the design and operation of the pilot facility are described in detail, as well as bench-top experiments that supplemented the pilot-scale filter runs. Next, the sampling and data collection protocols are summarized. Finally, the results from a wide range of tests are presented that were conducted to validate the performance of the pilot facility and associated bench-scale experiments. The detailed protocols for operating the pilot plant are provided in Appendix A.

2.1 DESCRIPTION OF PILOT PLANT

2.1.1 Pilot Plant Design

The pilot facility was designed to meet the specific research objectives of the FLEWR project, which required investigating five filter loading rates while keeping all other operational parameters constant. In addition, the pilot facility was designed to mimic the specific treatment train at the Monterey full-scale treatment plant as closely as possible. Thus, the filters were required to operate with constant head and constant flow rate. The choice of chemical coagulant and disinfectant was also dictated by the existing practices at the full-scale plant.

A photograph of the completed pilot plant, which was located inside an old chemical storage building, is shown in Figure 2.1. A detailed schematic of the pilot plant is shown in Figure 2.2, and a summary of the equipment used is provided in Table 2.1. Several photographs from the construction of the pilot plant are shown in Figure 2.6.

The pilot filters consisted of five identical columns that operated in parallel (see Figure 2.3 for a closer view of the filter columns). Filter columns consisted of clear PVC pipe (diameter = 20.3 cm), filled with anthracite (L = 1.2 m, d_{10} = 1.22 mm, UC = 1.34) and sand (L = 0.3 m, d_{10} = 0.62 mm, UC = 1.42). A sintered plastic bead under-drain supported the media (Leopold IMS Cap, Zelienople, PA). A weir controlled the water level above the top of the filter media at a constant 4.4 m. The loading rate through each filter was controlled by an electrically actuated butterfly valve (2.5 cm diameter Keystone EPI-TORC 3, Tyco, Princeton, NJ) and flow meter (Magflow MAG 1100, Danfoss, Milwaukee, WI) installed on the filter effluent line and configured in a feedback control loop. Seven pressure transducers (Dwyer Instruments, Michigan City, IN) located throughout the media bed (0.0, 7.6, 15, 61, 122, 130, and 152 cm from the top of the media) measured the pressure drop through the filters (see Figure 2.4). A rendered CAD drawing is provided in Figure 2.5; in this figure, the approximate water level during operation of the pilot plant is visible, as well as the two layers of media in the filter columns.

Secondary effluent from the full-scale treatment clarifier effluent well was pumped to the pilot plant. MS2 bacteriophage was seeded into the flow using a peristaltic pump and mixed using two inline static mixers (2″ diameter series 308 12-element, Koflo Corp., Cary, IL). Coagulant was then added using an additional peristaltic pump and mixed with four additional inline static mixers (see Figure 2.2). Coagulated flow entered a three-chambered flocculation tank with decreasing shear rates between chambers (100, 50, 32 s^{-1}), and 15.9 min theoretical detention at the typical pilot system flow rate of 3.3 m^3/h. Flocculation mixing was achieved with 30.5-cm diameter hydrofoil impellers (316SS XTF-3 high efficiency, Cleveland Eastern Mixing, Clinton, CT). This flocculated effluent was gravity fed to the filter columns.

Figure 2.1. Photograph of pilot filtration plant. The influent wastewater entered at the lower right, where addition of chemicals and MS2 coliphage occurred. The three transparent flocculation chambers are at the top right of the photograph. The transparent channel at the top left distributed the coagulated wastewater to the five filter columns on the left.

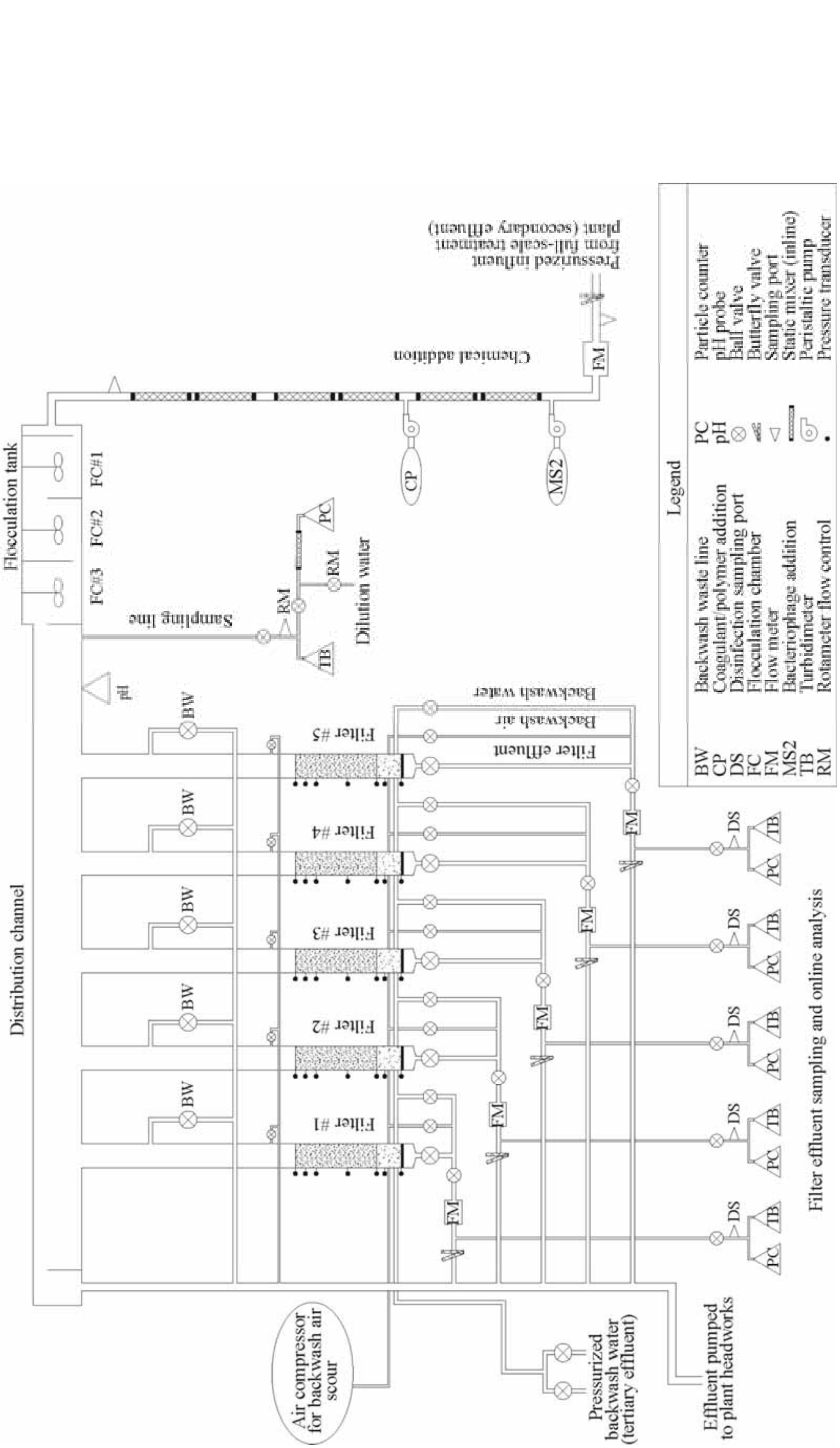


Figure 2.2. Schematic of pilot plant.

Table 2.1. List of Equipment Used in Pilot Filtration Plant

Equipment	Manufacturer	Model	Additional Specifications
Air Compressor	Craftsman Hoffman Estates IL	Model 919.167241	3 HP; 150 psi MAX; 4.9 scfm at 40psi, 3.5 scfm at 90 psi
Butterfly Valve	Tyco-Keystone Princeton NJ	EPI-TORC; EPI-3	Electric actuator; 1" valve
Chemical mixing (static mixer)	Koflo Corp. Cary, IL	Series 308	Inline; 2" diameter twelve-element PVC inline; two units between each addition point (MS2, ACH, and Polymer)
Flocculation Mixer	Cole-Palmer Vernon Hills IL	Servodyne high-speed, low-torque mixer system	3 – 180 RPM, 340 in-oz max continuous torque
Flow Meter	Danfoss Milwaukee WI	Magflo type MAG 1100, DN 15 and DN25	Electromagnetic; approx $\pm$ 2% error; system influent is 25 cm diameter, filter effluent flow meters are 15 cm
Media Support	Leopold Zelienople PA	IMS®	1" thick, removed from standard under-drain; pore volume 30 – 50%; HD polyethylene
Mixer Impeller	Cleveland Eastern Mixing Clinton CT	316SS XTF-3 High Efficiency	3 blade low shear hydrofoil impeller
Particle Counter	Hach Company Loveland CO	2200 PCX	Calibrated for 2 – 15 mm diameter particles; 8 size ranges with analog signal; flow rate is 100 mL/min through sensor
Peristaltic Pump	Cole-Palmer Vernon Hills IL	Masterflex computer controlled	1.6-100 RPM (0.10 – 6.00 mL/min); accuracy + 0.25%; Neoprene L/S® 13 tubing with 0.8 mm ID tubing
pH Meter	GLI Industries/Hach Loveland CO	Pro Series Model P3	Automatic pH calibration and temperature correction; accuracy $\pm$ 0.1% of span
Pressure Transducers	Dwyer Instruments Michigan City IN	Model 673-4	Range 0 – 10 psig (up to 23 ft of pressure head); accuracy $\pm$ 0.25%
Streaming Current Monitor	ChemTrac Systems Inc. Norcross GA	2000XR	Sample flow rate up to 5.0 gpm
Turbidimeter	Hach Company Loveland CO	1720D Low Range	Range: 0 – 100 NTU; Accuracy greater of + 2% of reading or + 0.02 NTU; sample flow 250 – 750 mL/min; calibrated with Formazin standard

Figure 2.3. Close-up photograph of pilot filter columns.

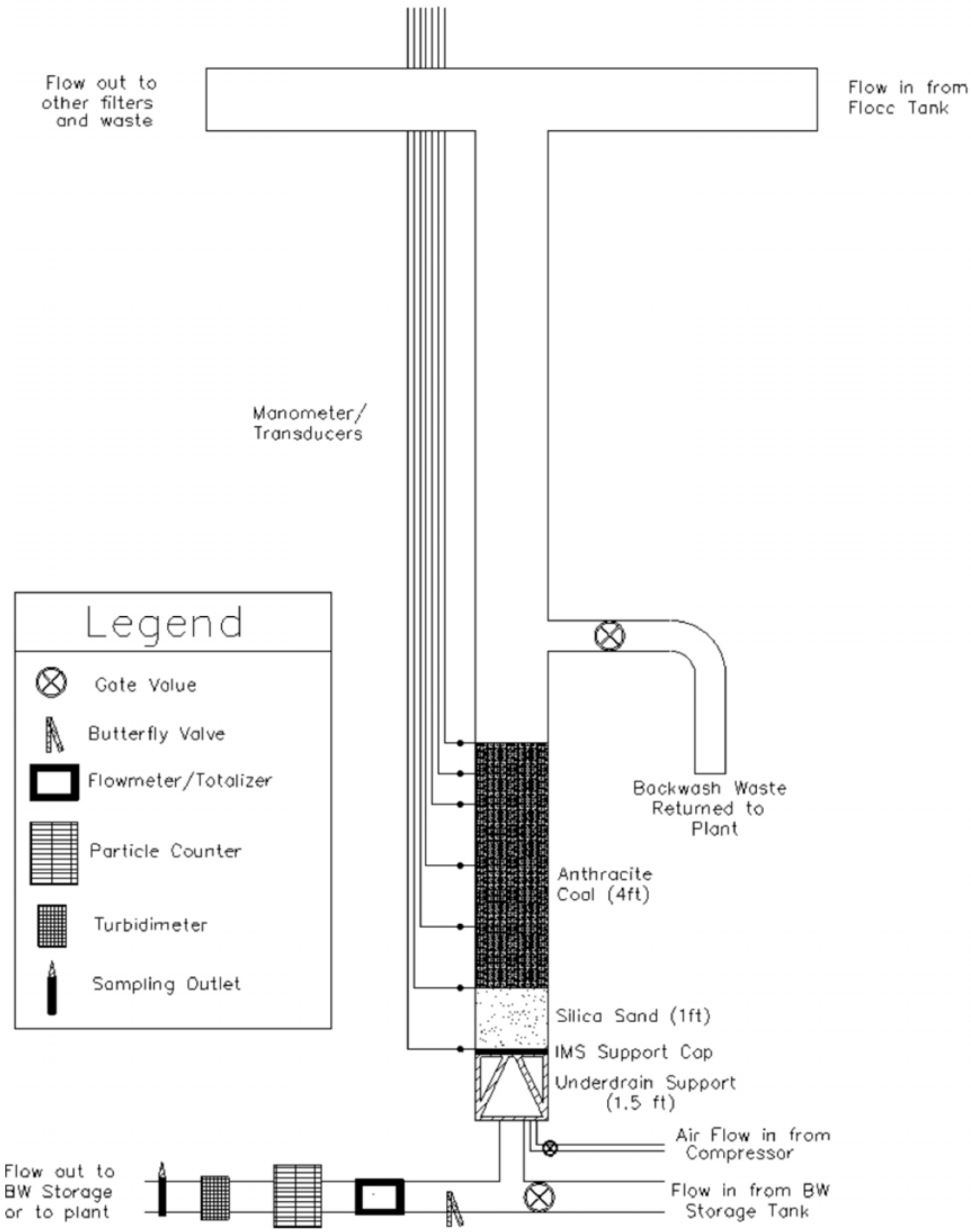


Figure 2.4. Schematic of individual filter.

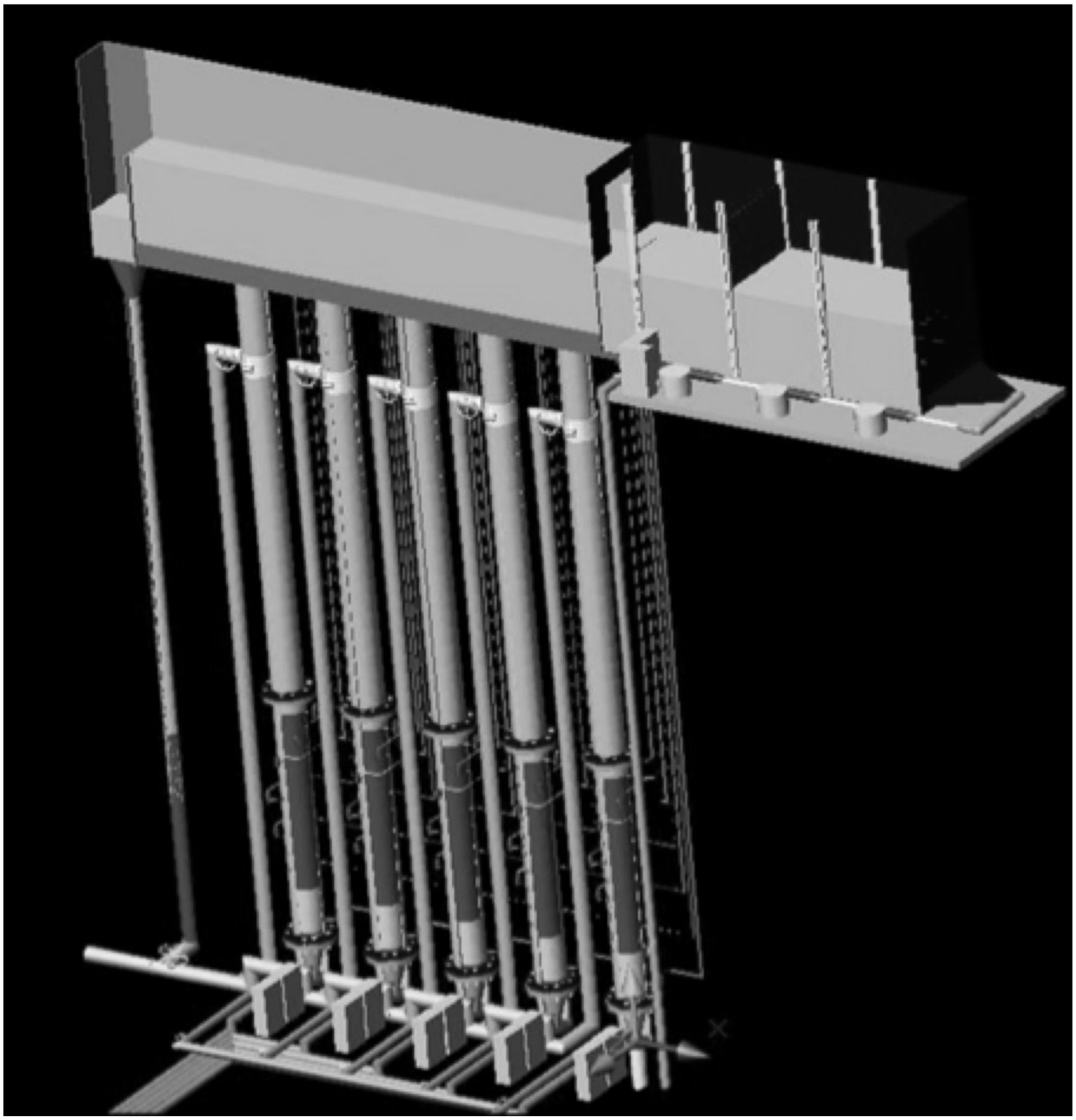
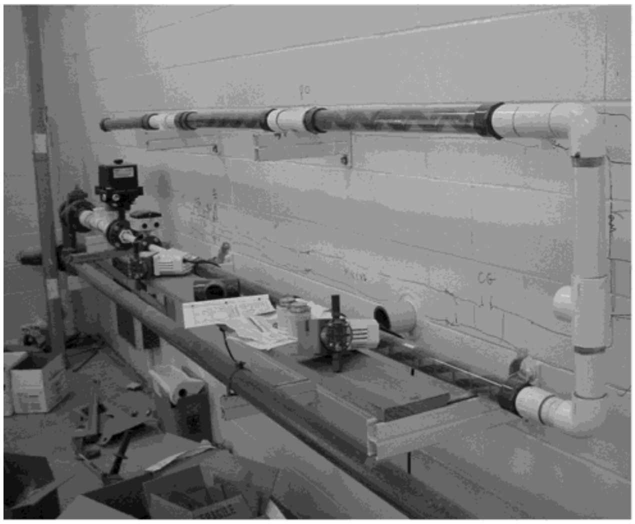


Figure 2.5. Rendered CAD drawing of filter pilot plant.



Influent Chemical Addition and Rapid Mixing

Initial Filter Column

Installing the Flocculation tank

Filter Effluent/Backwash piping

Effluent Flow Control and sampling ports

Figure 2.6. Photographs of pilot plant construction.

2.1.2 Operation of Pilot Plant

2.1.2.1 Filter Run Start Time

The secondary effluent from the full-scale treatment plant that was used to feed the pilot plant experienced significant diurnal variability. For example, the turbidity fluctuations over a 7-day period are graphed in Figure 2.7. Because we aimed to keep the influent quality constant across filter loading rates, this variability presented a challenge as the filter loading rates resulted in different run lengths. For example, the filter run loaded at 5 gal/ft^2-min typically lasted about 24 h, whereas the 7.5 gal/ft^2-min rate lasted about 17 h. Thus, if the filter runs started at midnight, the 7.5 gal/ft^2-min rate would experience more time with a lower influent quality than the 5 gal/ft^2-min rate.

To overcome this source of variability, the start times of the filters were staggered so that half of the runs started at the beginning of the day and half started at the end of the day. (Although a randomized start time would have been ideal from a statistical design perspective, it was not feasible due to staffing issues.) As presented in Chapter 3, the staggered start times resulted in all filter loading rates experiencing statistically equivalent average influent turbidity.

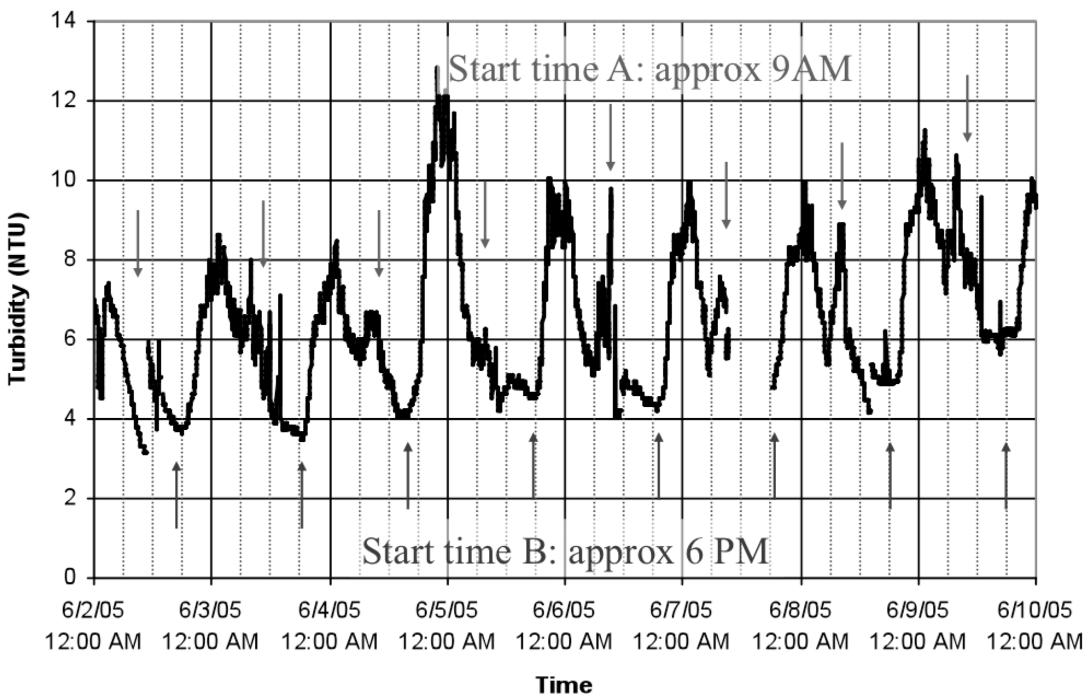


Figure 2.7. Secondary clarifier effluent turbidity at MRWPCA, June 2–9, 2005. Arrows identify the approximate filter start times that were chosen.

2.1.2.2 *Optimization of Chemical Dose*

During the first three sets of pilot plant filter runs, the coagulant dose was maintained at a constant level for each filter run. However, the coagulant had to be optimized for each run because of changes in influent water quality from day to day (e.g., compare 6/2/05 and 6/5/05 in Figure 2.7; seasonal changes were also very significant). Three factors were used to determine the appropriate coagulant dose for a filter run: (1) the dose and resulting effluent turbidity of the previous filter run; (2) the current dose in use at the full-scale facility; and (3) the start time of the filter run (a higher dose was used for runs initiated in the evening).

It is worth noting that several more objective approaches were investigated for choosing the coagulant dose, but none proved to be effective. A streaming current monitor was initially installed on the pilot plant influent line and was calibrated for selecting the coagulant dose, but it was not effective. Only major changes in zeta potential could be clearly detected by the streaming current monitor (e.g., no coagulant addition versus a high coagulant dose). In addition, changes in zeta potential did not correlate well with filter influent/effluent turbidity and coagulant dose (turbidity and coagulant dose would be steady, but streaming current would vary). Use of the streaming current monitor was discontinued after the Intensive Sampling Period 1 (see Chapter 3).

In addition, a modified laboratory jar test procedure was developed and performed before most runs during Intensive Sampling Periods 2 and 3. The procedure included adding several different coagulant doses[3] to the secondary effluent (1-L samples) in the jar test apparatus (PB-900 Programmable Jar Tester, Phipps & Bryd, Richmond, VA), followed by rapid (~17,000 s^{-1}) and slow mixing (5.3 min each of 82, 52, 38 RPM, which is an approximate shear rate of 100, 50, 32 s^{-1}). Then, 100 mL of each sample was added to the membrane filter apparatus (Whatman Grade 40 paper) and filtered until clogging occurred (up to 30 min under vacuum pressure). The filtrate turbidity and the time required to filter the sample were recorded; the impact of coagulant dose on filtrate turbidity from four of these tests is shown in Figure 2.8. The dose that resulted in the lowest filtrate turbidity without increasing filter clogging (i.e., the time to filter) was then designated as the optimum dose. However, the jar test results did not correlate well with average filter effluent turbidity, most likely because the coagulant dose was kept static for the entire filter run and the jar test only provided a limited snapshot of coagulation effectiveness (jar tests samples were collected between 8:00 and 10:00 a.m.). It became apparent that a more effective (and simpler) procedure for selecting the dose was to base the coagulant dose on recent experiences at the pilot plant (i.e., how well the previous dose worked) and experience at the full-scale plant (i.e., the average coagulant dose applied over the previous 48 h).

[3] Six coagulant blend (JC1679; aluminum chlorohydrate and poly-DADMAC cationic polymer) doses were selected and the concentrations varied between experiments. For most jar tests, the doses were between 0 and 30 mg/L. The pH was consistent throughout all jar testing and pilot testing, averaging 7.0 with all measurements falling between 6.6 and 7.3. The coagulant blend had minimal effect on the pH, where the addition of 30 mg/L coagulant blend led to a pH increase of less than 0.1 pH unit.

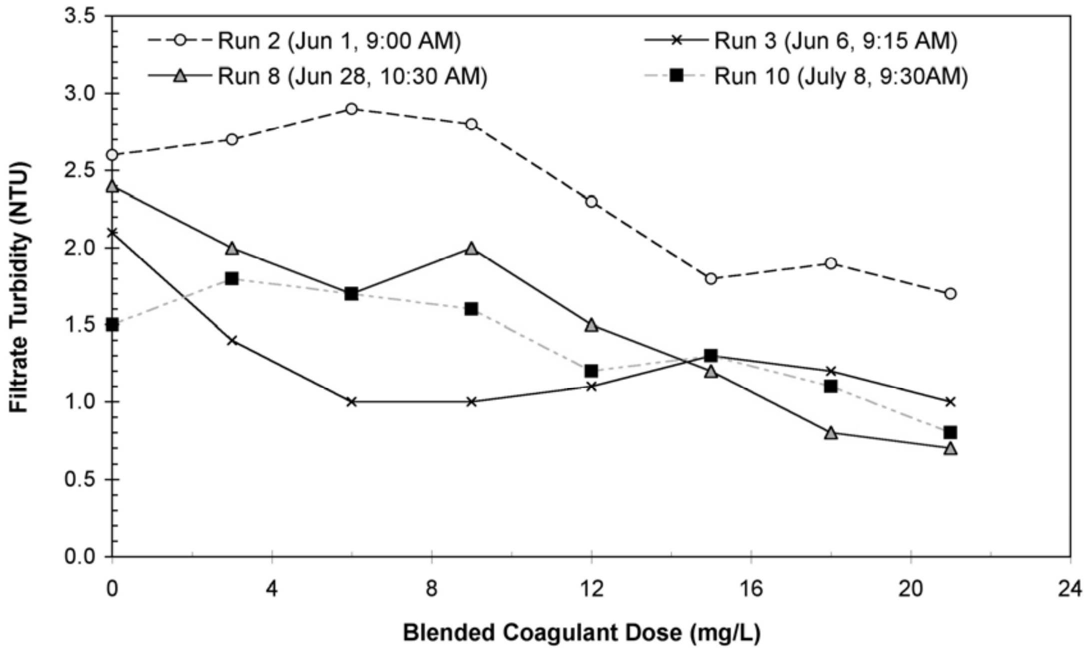


Figure 2.8. Select direct filtration jar test results conducted during ISP3.

Finally, a series of filter runs was conducted to test six coagulant doses at the five filter loading rates. As illustrated in Figure 2.9, increasing the coagulant dose from 18 to 22 mg/L resulted in a lower filter effluent turbidity, whereas increasing the dose from 22 to 26 or 30 mg/L resulted in higher effluent turbidities. Even higher coagulant doses of 40 and 60 mg/L caused effluent turbidities to decrease again, but were also associated with short filter run times. Thus, for the influent water quality during this set of filter runs, 22 mg/L appeared to be an optimal coagulant dose. It is also worth noting that the differences between loading rates were smallest when the coagulant dose was 18 mg/L, and the removal efficiency for turbidity was lowest.

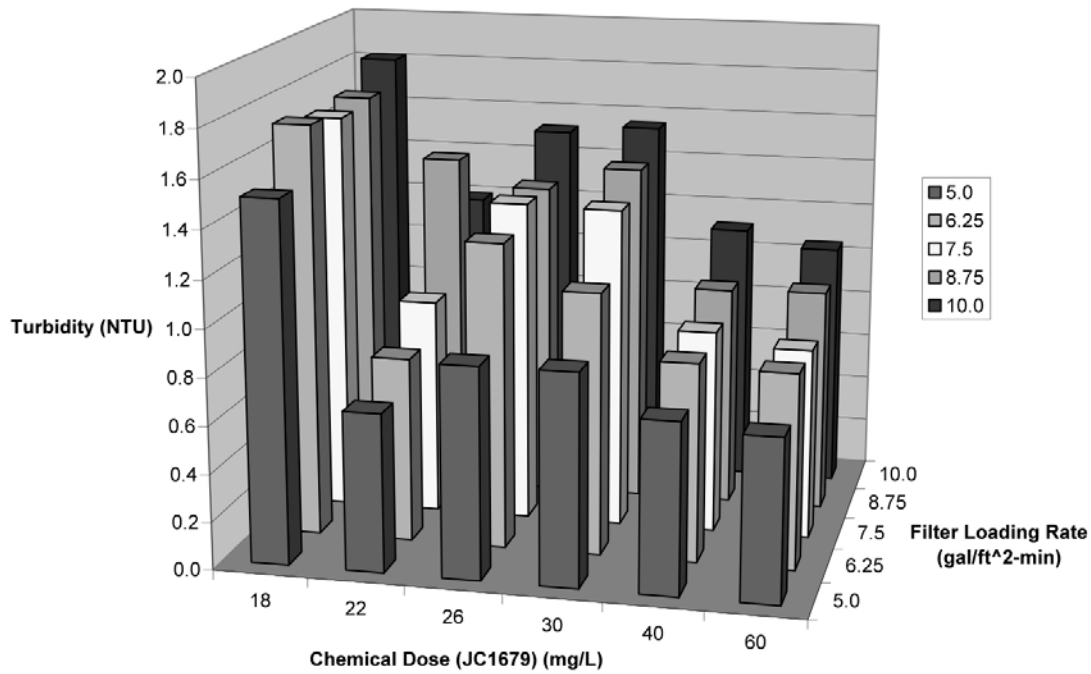


Figure 2.9. Average filter run turbidity by loading rate and coagulant dose.

2.1.2.3 Filter Backwashing

The pilot filters were backwashed after all filters reached a head loss of 3.4 m, which is the terminal head loss at the full-scale treatment plant. Backwashing included both an air scour (supplied by an air compressor) and high rate wash (supplied by full-scale tertiary-treated water). Air scour consisted of 1 min of air and a low-rate wash (12.2 m/h), followed by 3 min of air only. A high-rate wash (83.2 m/h) was then sustained for 20 min, causing 20% (0.3 m) fluidization of the media. At the completion of the backwash, the media was tapped down to the specified elevations (AWWA, 1992).

2.1.3 Data Collection

2.1.3.1 Continuous Measurements and Grab Samples

A summary of the data collected via continuous measurements and grab samples is provided in Table 2.2. All online instruments were connected to a SCADA system using 4-20 mA signal. Information from the online instruments was recorded once per minute. Additional information (not shown in Table 2.2) included flow rate, run time, and volume of water treated.

Online particles counters (PCX 2200, Hach Company, Loveland, CO) and turbidimeters (1720D Low Range, Hach Company) continuously monitored the flocculated filter influent and each filter effluent. Particle counts were collected in the size ranges 2–3, 3–4, 4–5, 5–7, 7–10, and 10–15 µm. The filter influent particle concentration exceeded the particle counter capacity (16,000 particles >2 µm per mL), so an inline system diluted the instrument flow with groundwater by a factor of 10. This inline dilution system consisted of rotameters (Blue-White Industries, Huntington Beach, CA) on both the sample and dilution water lines, regulating the flows prior to passing through a static mixer (0.75″ Series 308 12-element, Koflo Corp, Cary, IL). Particle counts on the influent wastewater were collected for 30 min prior to each filtration run. Once the filtration run started, the same particle counter was switched to sample the influent line postcoagulation.

An online pH meter (Pro Series 3, GLI Industries/Hach Company, Loveland, CO) measured the pH in the coagulated water. A streaming current monitor (SCM) was installed on the influent line during the first set of filter runs; however, the information was not found to be useful for optimizing the coagulant dose, so SCM data was not collected in subsequent experiments. Head loss data was collected via seven pressure transducers placed throughout each filter.

Grab samples for microbiological analysis were collected three times during each filter run in four locations: pre- and postbacteriophage addition, postflocculation, and postfiltration. Samples were analyzed onsite using the Colisure® method (using Quanti-tray® 2000, IDEXX Laboratories, Westbrook, ME) to determine concentrations of total coliform bacteria and *E. coli*. F^+ Male specific bacteriophage analysis was performed by BioVir Laboratories (Benicia, CA) on samples frozen with 5% TB Broth, using the double layer method (Adams, 1959) with host *E. coli* (ATCC 15597).

Table 2.2. Parameters for Grab Samples and Samples Measured Continuously[a]

Location	Turbidity	Particle counts	pH	SCM[b]	Head Loss[c]	Total Coliform	*E. coli*	MS2	Indigenous phage	Chlorine residual
Influent	C	C[d]		C		G	G	G	G	
Post-addition of MS2								G		
Post flocculation	C	C	C			G	G	G		
Filter effluent	C	C			C	G	G	G		
Disinfection						G	G	G[e]		G

[a] C = continuously measured and recorded once per minute; G = grab sampled.

[b] SCM = streaming current monitoring; data not useful and only collected during first testing period.

[c] Seven pressure transducers were placed throughout each filter bed to monitor head loss through specific sections of media.

[d] For 30-min prior to the start of each filter run, at which time counter was switched to monitor postcoagulation.

[e] Dropped after first test period, because chloramine disinfection of MS2 found to be ineffective.

2.1.3.2 Disinfection Batch Tests

Impacts of filter loading rate on subsequent disinfection were evaluated using batch experiments. Filter effluent samples were collected simultaneously from all filters between 3 and 4 h after start-up. Sodium hypochlorite was added to 1-L samples of filter effluent to determine the coliform inactivation at CT values up to 1200 mg-min/L (10 mg-Cl/L residual at 120 min). Using the Colisure® method, total coliform bacteria and *E. coli* concentrations were determined at multiple time points throughout the disinfection for each sample, to determine inactivation kinetics. Total chlorine concentrations were determined using the iodine titration method (Cleasby et al., 1998) modified by diluting iodine (in Milli-Q water) to 0.00282 N, to increase method resolution. Only minimal nitrification occurred at the full-scale treatment plant, so it is believed that all of the chlorine was rapidly converted to chloramines (breakpoint chlorination was not practiced at the full-scale plant, nor in these experiments). To determine the effect of large particle aggregates on the tailing observed in the disinfection curves, a randomly selected subset of the disinfection samples was filtered through 5-μm pore membrane filters (Nuclepore® polycarbonate, Whatman Company, Brentford, UK) prior to chlorination.

2.1.3.3 Data Analysis

The mean values for the continuous data (turbidity, particle counts, and head loss) from each filter run were determined by averaging all data points collected throughout each filter run (N ~ 700-1300, depending on run length). Overall means for each parameter were calculated by averaging the individual run means for each test period (N ~ 10 for each loading rate).

For the indicator organisms collected by grab samples, the values from all filter runs in a test period were averaged to determine the overall mean (N ~ 30). The arithmetic mean (turbidity and particle counts) or geometric mean (indicator organisms) was used, depending on whether the distribution approximated either normal or log normal. Efficiencies for all parameters were stated as the fraction removed (1- effluent concentration/influent concentration). All uncertainties are reported at the 95% confidence level; for normally and log normally distributed parameters, the confidences are written as *mean ± confidence limit* and *mean (confidence range)*, respectively.

2.2 VALIDATION OF PILOT PLANT PERFORMANCE

2.2.1 Filter Replicates

Several tests were undertaken to ensure that the five filters performed as replicates. The first test was on the size distribution of the filter media. The filters were loaded with media collected as cores from the full-scale filters in October 2003. After 19 pilot-system runs, the size distribution of the pilot filter anthracite media, as well as the original anthracite media that had been stored, was measured in January 2005. As seen in Table 2.3, the size distribution and uniformity coefficient of the original anthracite media, and the media in each filter, was comparable. The size distribution of the sand media was not analyzed because it could not be extracted from the pilot filters nondestructively; it was also believed that the anthracite would have been more sensitive to air scouring.

Table 2.3. Comparison of Anthracite Grain Size Distribution Between Pilot Filters (January 2005) and Media Prior to Pilot-Plant Start-Up

Source	D_{60} (mm)	D_{10} (mm)	Uniformity Coefficient
Full-scale filter cores	1.22	1.64	1.34
Filter 1	1.24	1.64	1.32
Filter 2	1.23	1.65	1.34
Filter 3	1.26	1.66	1.32
Filter 4	1.22	1.64	1.34
Filter 5	1.23	1.65	1.34

On several occasions, all five filters were operated at the same filter loading rate. The results of one test (with variable coagulant dose) are shown in Figure 2.10. All five filters produced effluent turbidities that were nearly identical. During these pilot filter runs, one full-scale filter was also operated at a constant loading rate of 5 gal/ft^2-min. The pilot filter effluents were about 0.5 NTU lower than the full-scale turbidity, and the overall shape of the turbidity curves was similar. A possible explanation for the higher turbidity at the full-scale plant is that coagulant dosing was not as precise at the full-scale facility, such that the full-scale filters received a lower coagulant dose.

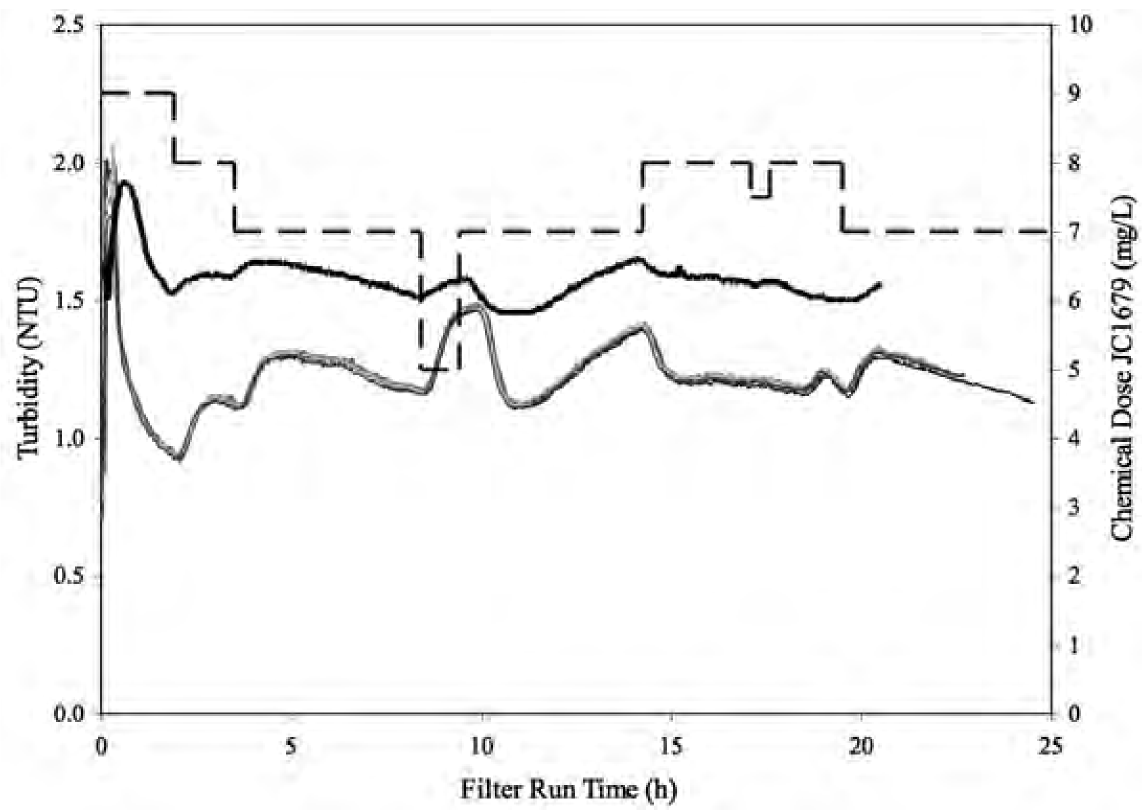


Figure 2.10. Comparison of effluent turbidity between all five pilot-filters with one of the MRWPCA full-scale filters with a variable coagulant (dashed line with units on right axis). July 22–23, 2005.

2.2.2 Flow Control

Because the focus of this study was on filter loading rate, maintaining constant flow, and characterizing the flow rate through the filter columns were major considerations. As discussed in Section 2.1.1, the loading rate was controlled by an electrically actuated butterfly valve and flow meter installed on the filter effluent line and configured in a feedback control loop (similar to the full-scale filters). The measured flow rate during one set of filter runs is shown in Figure 2.11. Although the mean flow was constant throughout a run, significant fluctuations occurred, and the flow control system for each filter column resulted in a unique flow pattern. Any differences between filter columns were addressed by rotating the loading rates through the filter columns in a randomized order. It was believed that the fluctuations in loading rate may have caused decreased filter performance because of hydraulic pulses and to periodically higher loading rates.

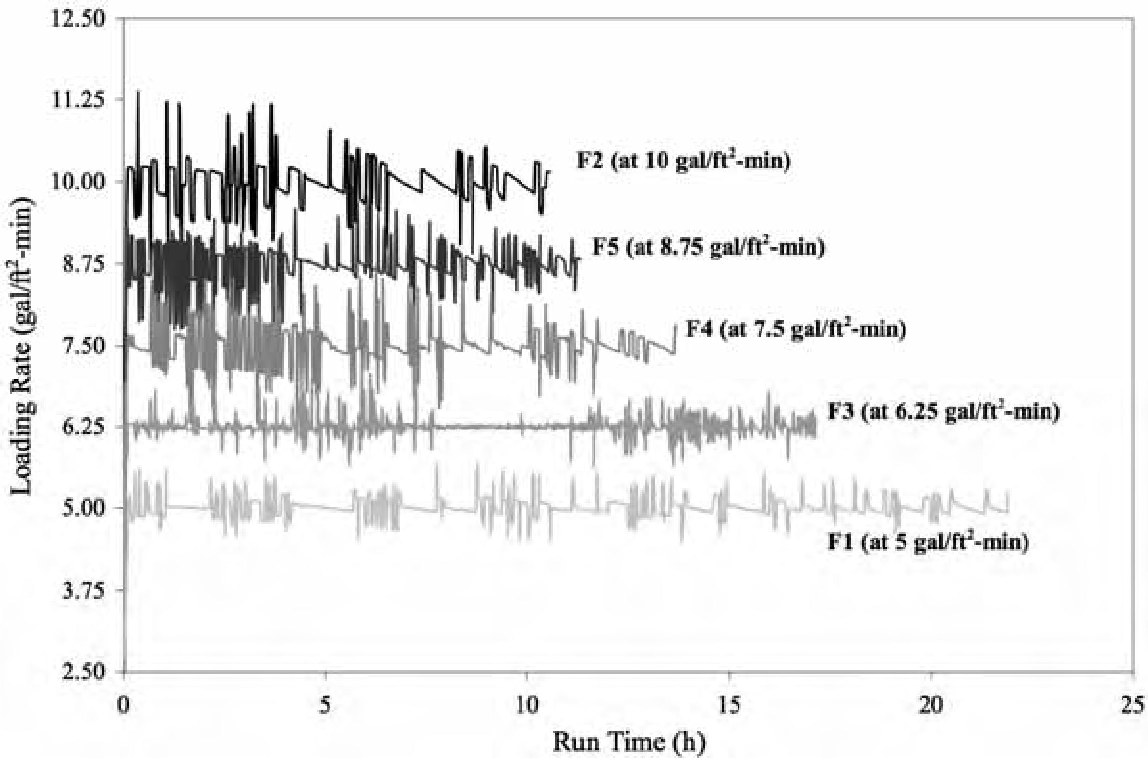


Figure 2.11. Pilot-scale variability in filter loading rate during one filter run. Note that some filters (2, 4, and 5) were more variable than others (Filters 1 and 3).

As shown in Figure 2.12, the variability at the pilot-plant filters was higher than at the MRWPCA full-scale filters. Although median values for both systems were 5.0 gal/ft^2-min, the full-scale system had better flow control, with loading rates ranging from 4.80 to 5.11 gal/ft^2-min, compared to the pilot system, which ranged from 3.97 to 5.86 gal/ft^2-min. Flow control at the pilot scale was similar at the 5.0 and 7.5 gal/ft^2-min rates when normalizing the data by the rate (data not shown). Thus, it was concluded that the pilot filters provided a conservative model of the full-scale filters in terms of flow control.

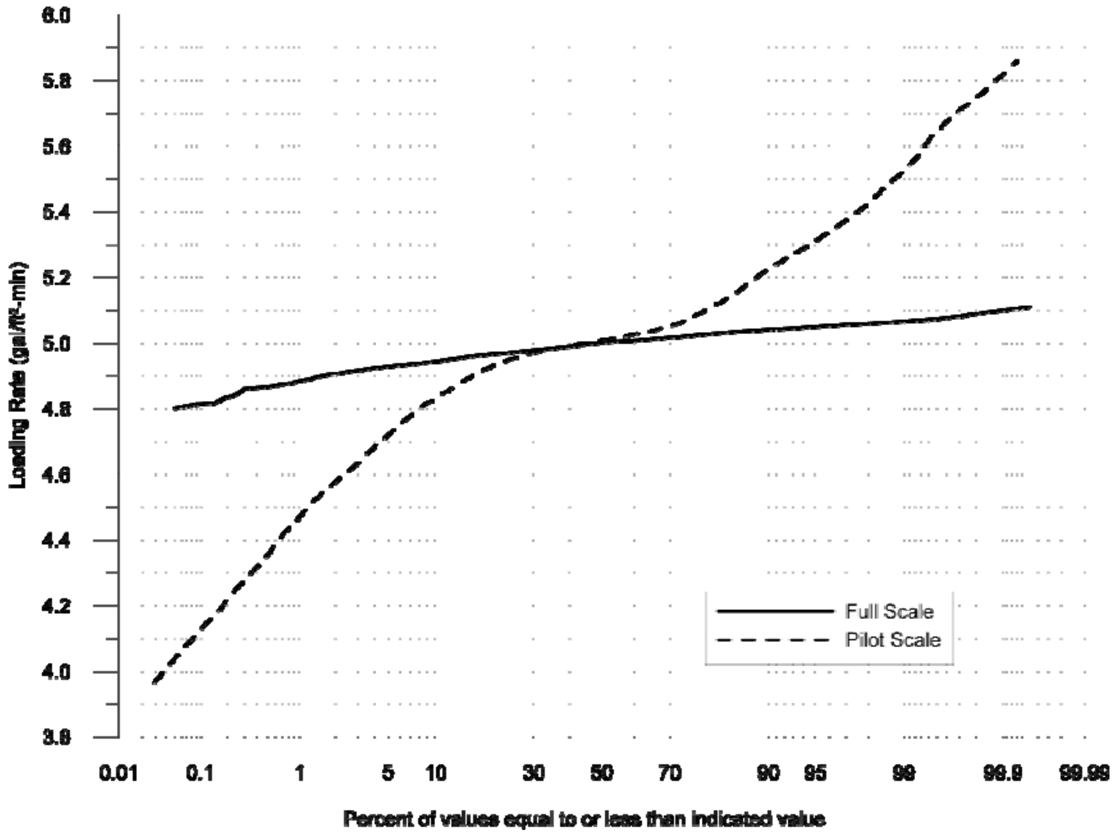


Figure 2.12. Variability in pilot-scale filter flow control at 5 gal/ft^2-min compared with MRWPCA full-scale filter during pilot-/full-scale comparison runs conducted on May 18, 2005 and July 22, 2005.

2.2.3 Microbiological Sampling

Several factors were considered in determining when to collect grab samples for microbiological analysis, including the travel time, ripening, and the potential presence of indigenous coliphage in the pilot plant influent wastewater.

Because the filter influent wastewater experienced significant variability in water quality, the effluent grab samples were collected at the mean travel time after the influent sample was collected. The travel times were determined using an analysis of the hydraulic residence time distribution. The results from a theoretical analysis were compared with a tracer study using a step input of the conservative tracer LiCl.

The theoretical analysis modeled flow through the pilot plant pipes and filters as ideal plug flow and modeled each of the three flocculation chambers as a complete mixed tank reactor (Levenspiel, 1999). In Figure 2.13, the cumulative age distribution after each of the flocculation chambers is shown for the model and the tracer study. As expected, the mixing in each sequential flocculation chamber caused the flow to become more dispersed (i.e., there was a wider distribution of residence times about the mean). There was excellent agreement between the model and tracer study results.

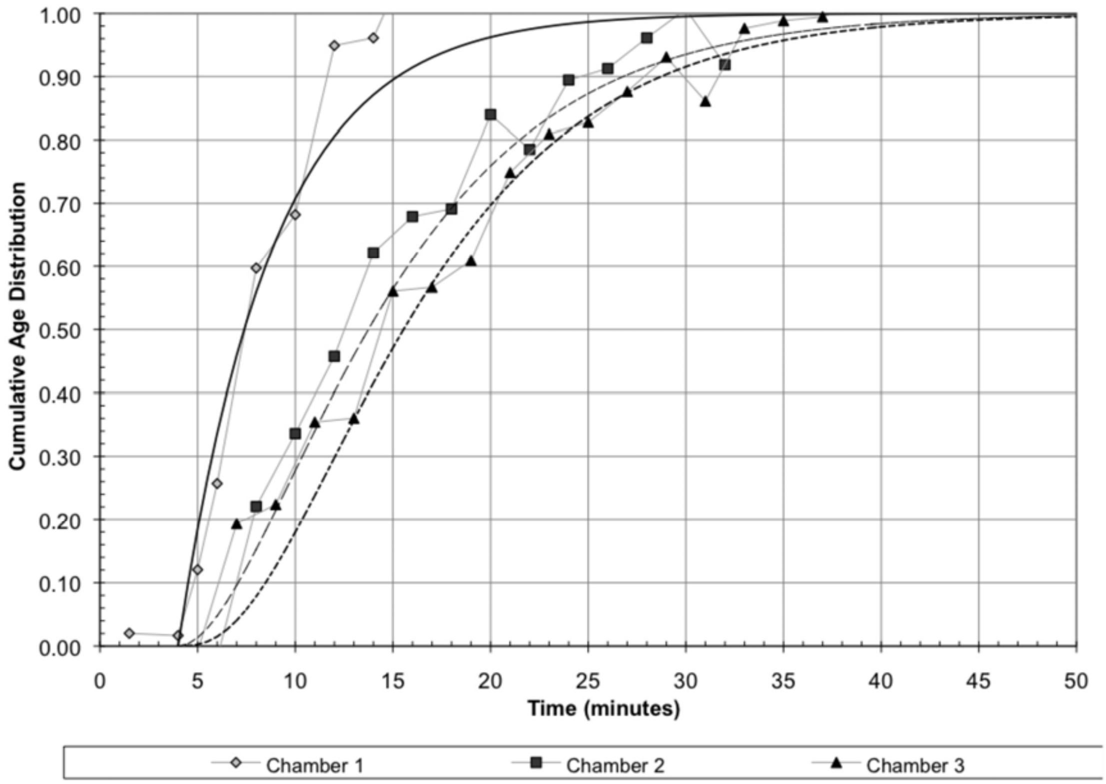


Figure 2.13. Cumulative age distribution determined from the LiCl tracer study (symbols) and from theoretical analysis (curves) for each chamber of the three-chambered flocculation tank.

After the third flocculation chamber, the flow was distributed to the five filter columns operating in parallel. In Figure 2.14, the cumulative age distribution after each filter column is shown. As expected, some mixing occurred in the filter columns causing the age distribution to spread out compared to the model, which assumed ideal plug flow through the columns. The five filter columns appeared to operate similarly to each other. Nonetheless, to account for any variability in hydraulic flow in the experimental design, filter loading rates were rotated through the different filter columns in a randomized order.

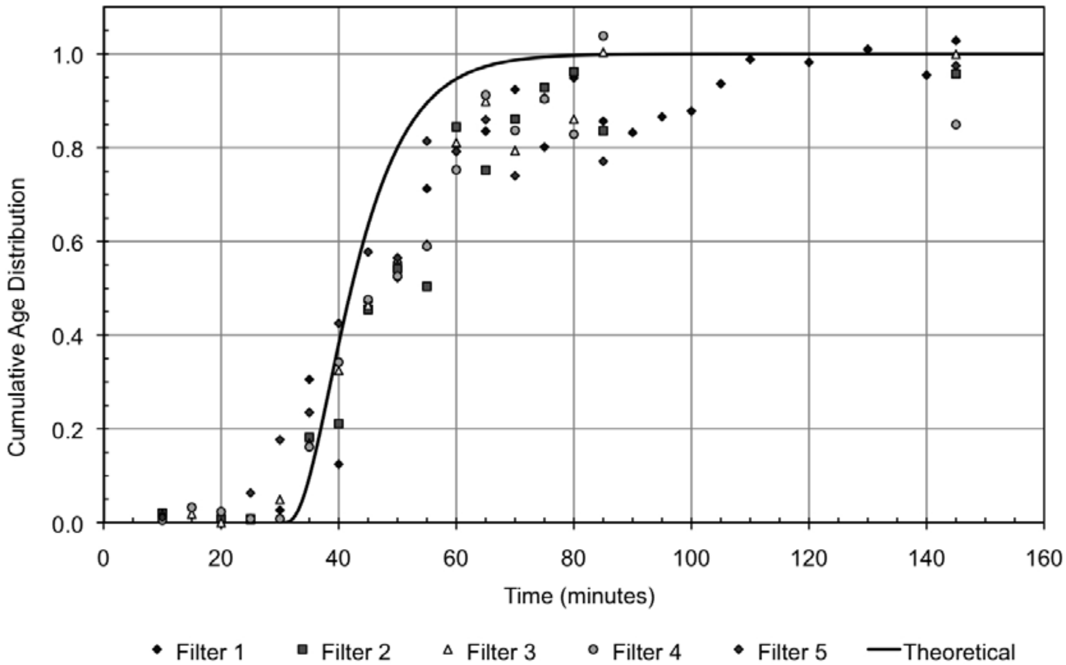


Figure 2.14. Cumulative age distribution through the five pilot-filter columns as observed through LiCl tracer study (symbols) compared with the theoretical distribution (curve).

The measured and predicted mean hydraulic residence times (HRT) after each flocculation chamber, and after each filter, are compared in Table 2.4. There was good agreement between the model and the tracer study results. Therefore, the model results were used to determine the sampling times for the grab samples.

Table 2.4. Summary of Measured Versus Predicted Mean Hydraulic Residence Time (HRT) Through the Pilot System

Location	Measured mean HRT (min)	Predicted mean HRT (min)	Difference (min)
Flocculation Chamber 1	4.7	5.3	-0.6
Flocculation Chamber 2	11.3	10.6	0.7
Flocculation Chamber 3	12.9	15.9	-3
Filter 1	56	49	7
Filter 2	49	49	0
Filter 3	50	49	1
Filter 4	48	49	-1
Filter 5	47	49	-2

Note. The flocculation chambers operated in series, whereas the filters operated in parallel.

During one set of filter runs, grab samples were collected with high frequency to determine if there was any impact of ripening on removal of total coliform bacteria, *E. coli*, and seeded MS2 coliphage. As shown in Figures 2.15 and 2.16, the concentrations of MS2 and total coliform bacteria in filter influents and effluents were highly variable. The *E. coli* plot was very similar to the total coliform plot (data not shown). No significant effect of ripening (defined as a change in the removal rate over time) was observed. Therefore, it was determined that grab samples should be collected at three intervals spaced throughout the filter runs. The collection time for the secondary effluent samples was 30, 150, and 180 min after filter start-up. The coagulated samples and filter effluent samples were collected 14 and 50 min after the secondary effluent sample, respectively, based on the tracer study results.

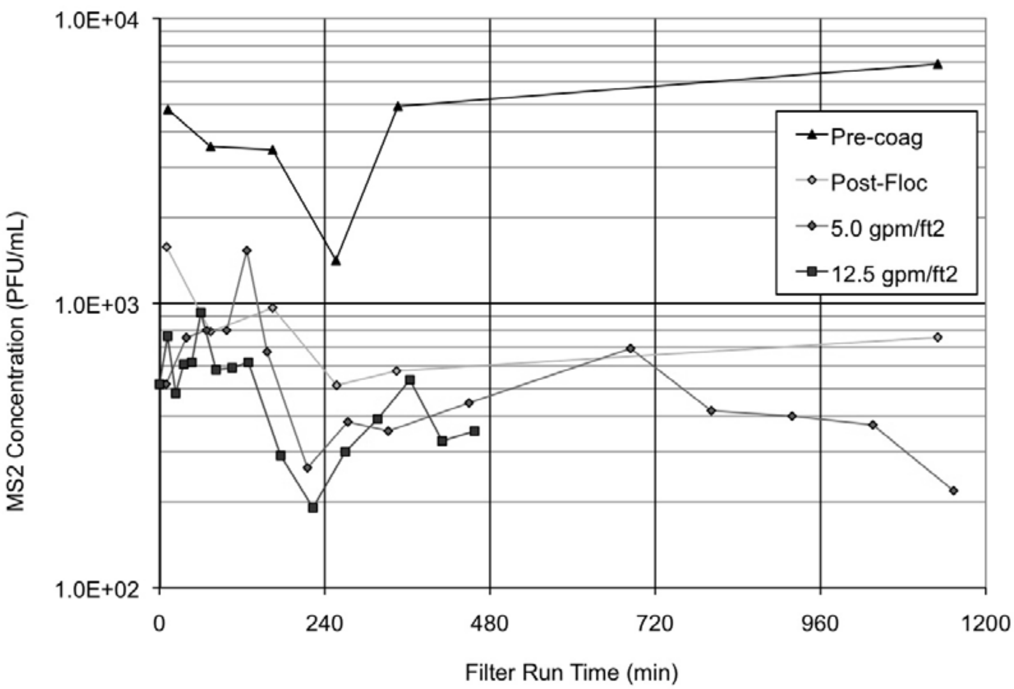


Figure 2.15. Test for effect of filter ripening on MS2 removal through pilot filter. Precoagulated and postflocculated times are adjusted to match the corresponding filter effluent.

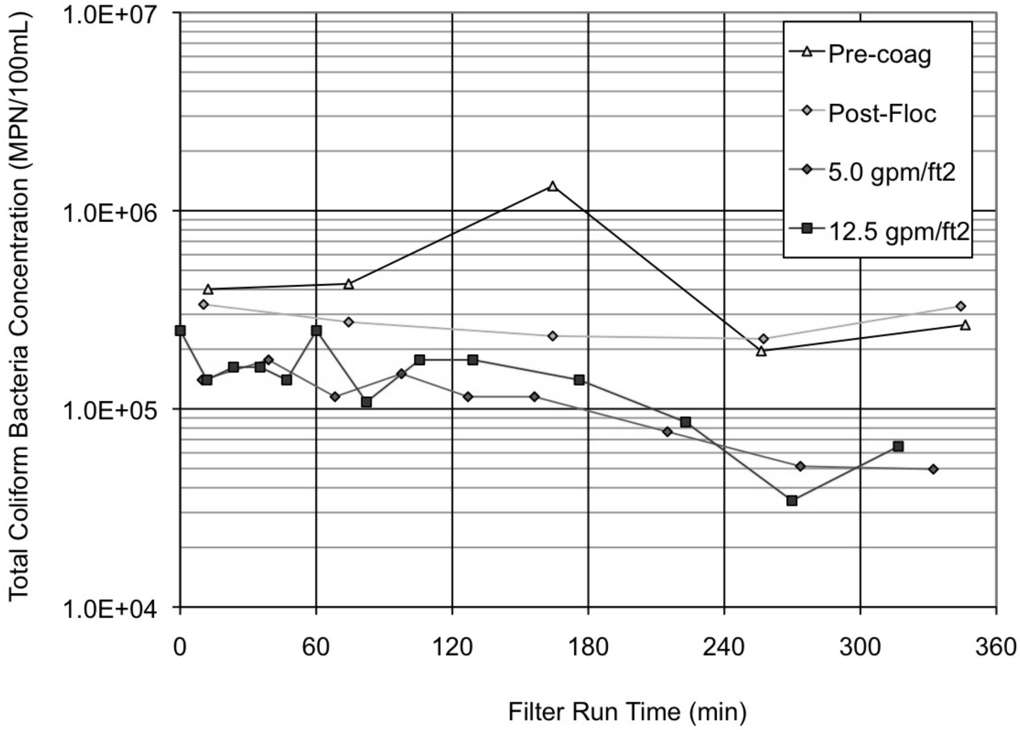


Figure 2.16. Test for effect of filter ripening on total coliform bacteria removal through pilot-filter. Precoagulated and postflocculated times are adjusted to match the corresponding filter effluent.

The concentration of indigenous F+ male specific and somatic coliphage were also monitored at the beginning of the study, as reported in Table 2.5. The median and maximum concentrations of F+ coliphage, 7 and 180 PFU/mL, respectively, were much lower than seeded concentrations (~10^5 PFU/mL).

Table 2.5. Indigenous Male Specific F+ and Somatic Coliphage During Pilot Testing

Metric	F+ Coliphage (PFU/mL)	Somatic Coliphage (PFU/mL)
No. of samples	83	20
Mean (of positive samples)	16.05	0.85
Median	7	0.25
Minimum	<1.0	<1.0
Maximum	180	3.5

2.2.4 Filter Backwash Effectiveness

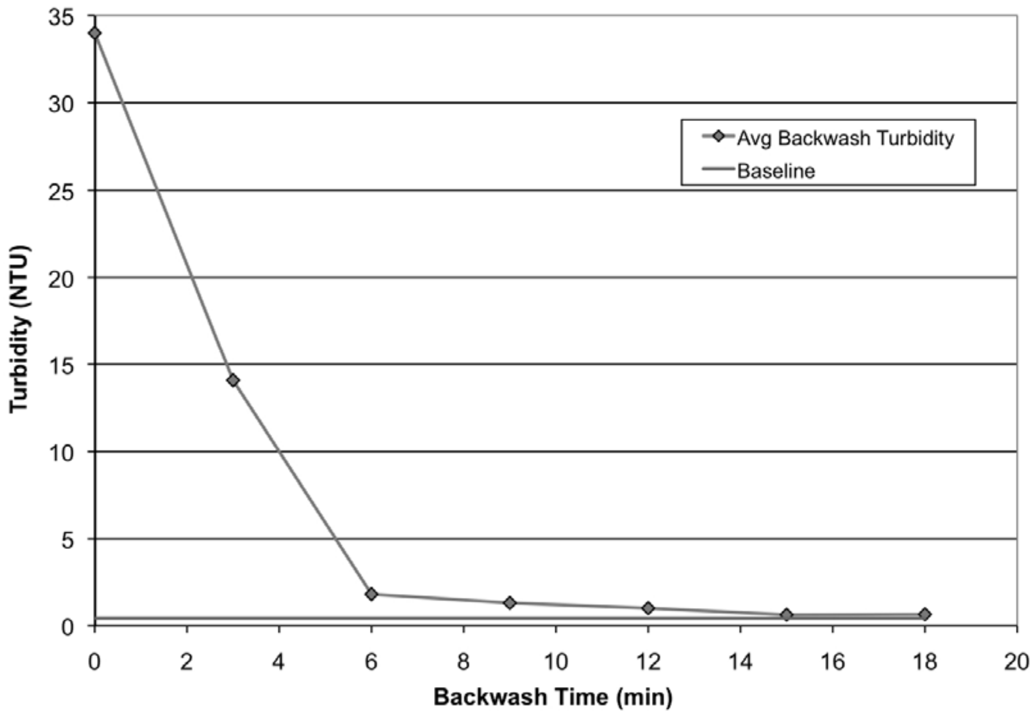


Figure 2.17. Turbidity of backwash water as function of high-rate backwash time (diamonds). Baseline turbidity is the average turbidity of the tertiary effluent used for backwashing. Turbidity values are average from all backwash characterization tests from all pilot-filters (*n*=9).

The filter backwash procedure included an air scour period followed by a high-rate backwash with water only (see Section 2.1.2.4). To ensure that the length of the high-rate water backwash was adequate, two experiments were conducted in which the turbidity of the backwash water was monitored during the high-rate portion of the backwash. As shown in Figure 2.17, after approximately 15 min, the turbidity of the backwash water was indistinguishable from the tertiary effluent used for backwashing. Thus, the 20 min, high-rate backwash time was found to be sufficient to ensure removal of particles that were scoured from the media.

Another metric that was used to determine the adequacy of backwashing throughout the study was to monitor the clean bed head loss (defined as the head loss during the first 30 min of a filter run, once the target filter loading rate was reached, typically within 5 min). In Figure 2.18, the clean bed head loss is shown for the five different loading rates for the first 10 filter runs. Because the clean bed head loss did not increase after consecutive filter runs, it was concluded that the backwashing was effective. We continued to monitor clean bed head loss throughout the study.

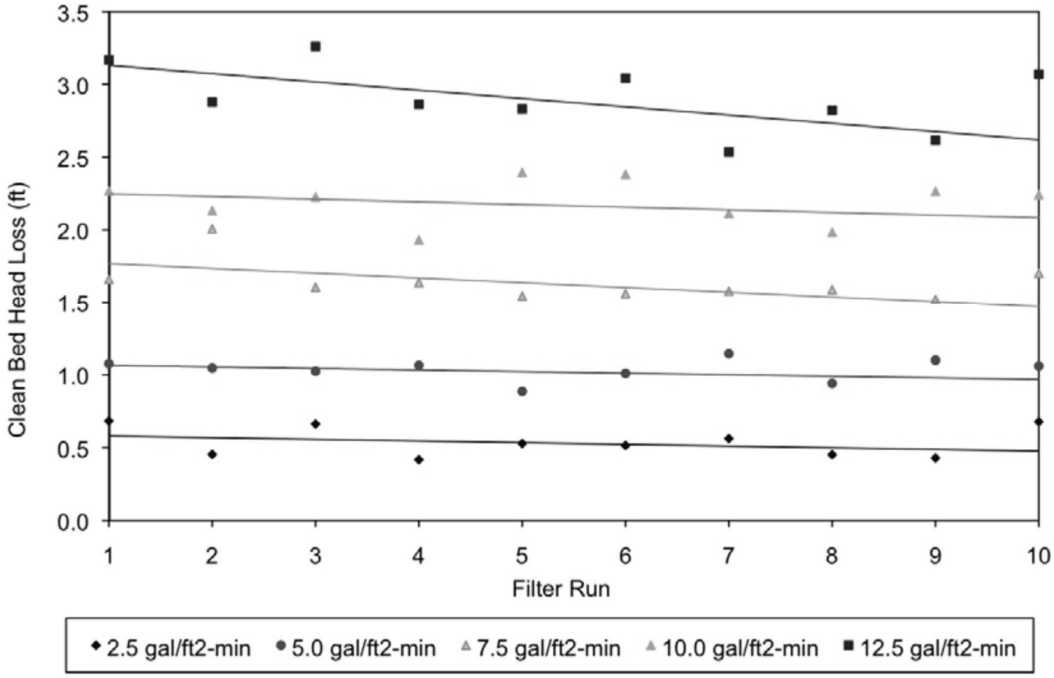


Figure 2.18. Clean bed head loss over the first ten pilot filter runs.

CHAPTER 3

IMPACT OF FILTER LOADING RATE AT THE PILOT PLANT

3.1 SUMMARY OF TESTING PERIODS

Four different sets of filter runs were conducted at the pilot filtration facility (Table 3.1; a complete list of all pilot filter runs is provided in Appendix B). During the first three intensive sampling periods (ISP), five loading rates were tested simultaneously and all loading rates received the same coagulant dose. This configuration was optimal for ensuring that all filter loading rates experienced the same influent quality and for isolating the impact of filter loading rate. During each period, the five loading rates were tested at least twice on each filter column (~10 runs at each rate in a randomized order), which eliminated any bias that was due to minor differences between filters as well as instruments.

During ISP1, the operation and data collection protocols were refined and several key changes were made. The filter loading rates studied during ISP1 were 2.5, 5.0, 7.5, 10.0, and 12.5 gal/ft^2-min. After completion of ISP1, it was decided to drop the 12.5 gal/ft^2-min rate because of the short run times achieved (see Section 3.2). The 2.5 gal/ft^2-min was also dropped, in order to focus data collection on rates higher than the current regulatory limit of 5 gal/ft^2-min. Thus, during ISP2 and ISP3, the loading rates tested were 5.0, 6.25, 7.5, 8.75, and 10.0 gal/ft^2-min. The coagulant was also changed after ISP1, from our own blend of aluminum chlorohydrate (ACH) and cationic polymer, to JC 1679, which is a proprietary blend of ACH and poly-DADMAC cationic polymer (approximately 0.2 mg-Al/mg-coagulant blend; JenChem Inc., Walnut Creek, CA). The main reason for the change was to mimic the full-scale plant, which also changed chemicals. However, the use of a single chemical also simplified the chemical optimization and pilot plant operation.

Table 3.1. Summary of Testing Periods at the Pilot Filtration Facility

	ISP1	ISP2	ISP3	SRO
Dates	Aug 17 - Sep 7 2004	Mar 7- Mar 29 2005	May 25 - Jul 15 2005	Sep 12 - Oct 7 2005
No. of Runs	11	10	12	10
Coagulant	ACH (10-15 mg/L)	JC 1679 (16-30 mg/L)	JC 1679 (9-13 mg/L)	JC 1679 (0.5-8.5 mg/L)
Polymer	Cationic Polymer (0-1.0 mg/L)			
Loading Rates	2.5 5.0 -- 7.5 -- 10.0 12.5	-- 5.0 6.25 7.5 8.75 10.0 --	-- 5.0 6.25 7.5 8.75 10.0 --	-- 5.0 -- 7.5 -- -- --

ISP2 and ISP3 provided robust data sets for characterizing the impact of filter loading rate on treatment performance. Because of improved influent water quality, the chemical doses used during ISP3 were lower than during ISP2. A key finding from ISP2 and ISP3 was that an increase in filter loading rate decreased the removal efficiency of the filters. Thus, in the fourth test period, named single rate optimization (SRO), only two filter loading rates were studied. The coagulant dose was continually optimized by the pilot plant operator such that the average effluent turbidity for each run was 1.90 NTU (corresponding to the treatment goal at the full-scale plant). The resulting coagulant doses were lower, primarily because the dose could be lowered as the influent water quality improved. In the SRO configuration, all five pilot filters operated at the same loading rate during a run, and the runs were alternated between the two loading rates.

An important criterion for data analysis was the definition of the length of a filter run. In most cases, the end of a filter run was determined by reaching maximum head loss. In some cases, the maximum acceptable turbidity was reached before maximum head loss. To be consistent with the California Water Recycling Criteria, the maximum turbidity was defined as an average effluent turbidity of 2.49 NTU during the ISP runs (see Section 1.2; a turbidity value of 2.49 NTU is interpreted as being equivalent to the single digit “2 NTU” specified in the regulations). During the SRO runs, because the average effluent turbidity never exceeded 2.1 NTU, the end of a filter run was determined either by reaching terminal head loss or after a maximum run time of 24 h (significantly longer run times were not feasible from an operational perspective).

3.2 DETAILED RESULTS FROM ONE SET OF FILTER RUNS

In this section, detailed plots from one set of filter runs are provided. A more detailed understanding of the characteristics and key variables affecting the filter runs is helpful before presenting summary results from each of the pilot test periods in subsequent sections. The following graphics are from run 12 of the third set of pilot filter runs (ISP3). The coagulant (JC1679) was 10.3 mg/L, the filters were started on July 14, 2005 at 17:56, and the last filter to shutdown (5 gal/ft2-min) was on July 15 at 15:15. During this run period, the average influent turbidity ranged from 6.2 to 6.9 NTU, resulting in higher average influent turbidities as the loading rate increased. The average filter effluent turbidities were 0.81, 1.05, 1.28, 1.59, and 1.79 NTU for 5, 6.25, 7.5, 8.75, and 10 gal/ft^2-min.

In Figure 3.1, the head loss accumulation throughout the filter runs is shown for the five loading rates. At time zero, the increase in clean bed head loss is visible for the higher loading rates. After time zero, the head loss increased faster for higher loading rates because of the larger volume of water treated. All of the filters shut down because they reached terminal head loss.

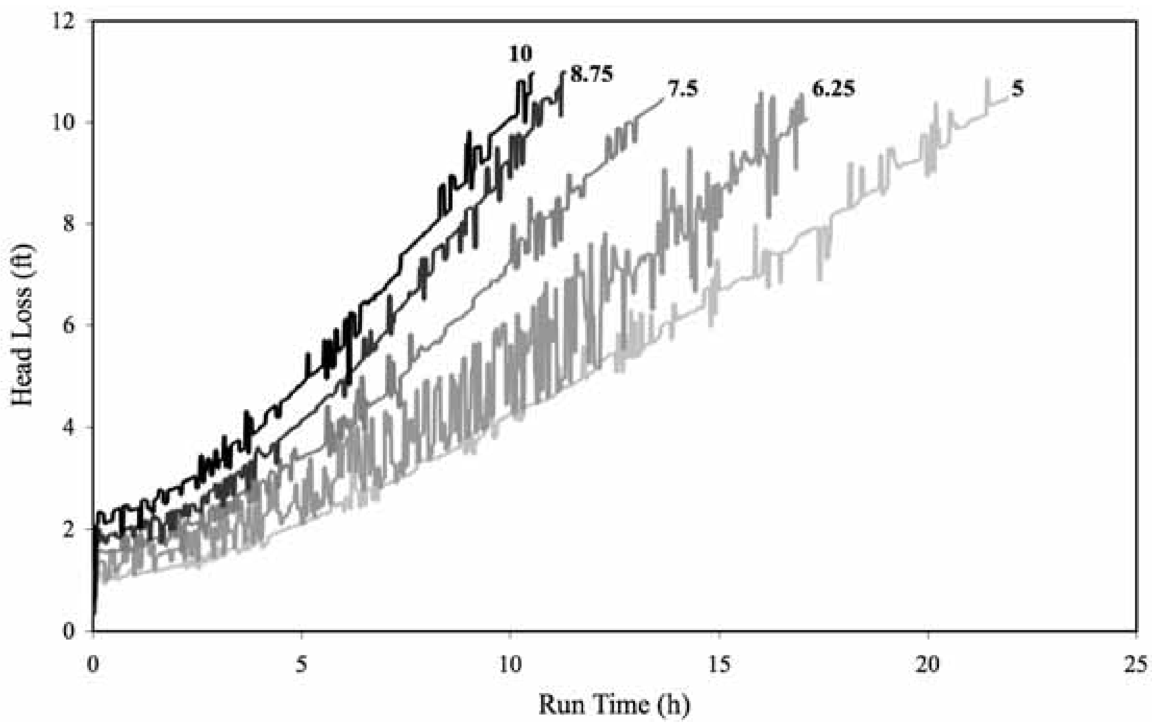


Figure 3.1. Pilot-scale build-up for head loss by loading rate during an example filter run for loading rates 5–10 gal/ft^2-min.

The turbidity of the precoagulated filter influent (secondary effluent), the coagulated filter influent, and the five filter effluents as a function of run time is shown in Figure 3.2. Note that the turbidity increased because of coagulation, as a result of floc formation. The initial spike in filter effluent turbidity was a result of backwash water being flushed from the filter columns; data from this period were not included in the analyses. The first influent water exited the filter columns after about 30 min (see Figure 2.13). No filter ripening period was observed, possibly because of the high particle content of the influent. As evident in Figure 3.2 and the closer view in Figure 3.3, during most filter runs the effluent turbidities increased gradually throughout the run. However, the effluent turbidities also fluctuated in response to changes in the influent turbidity. Comparable plots for particles in the 2–5 and 5–15 μm size ranges are presented in Figures 3.4 to 3.7.

An alternative to visualizing the filter effluent quality as a function of filter run time is to look at the cumulative volume of water treated. In Figures 3.8 and 3.9, the effluent turbidity and effluent particle counts in the 2–5 μm range are plotted versus the volume treated. If equivalent removal was achieved by all filter loading rates, one would expect the effluent curves to collapse on top of each other. However, as seen in the plots, the effluent quality decreased gradually as the filter loading rate increased.

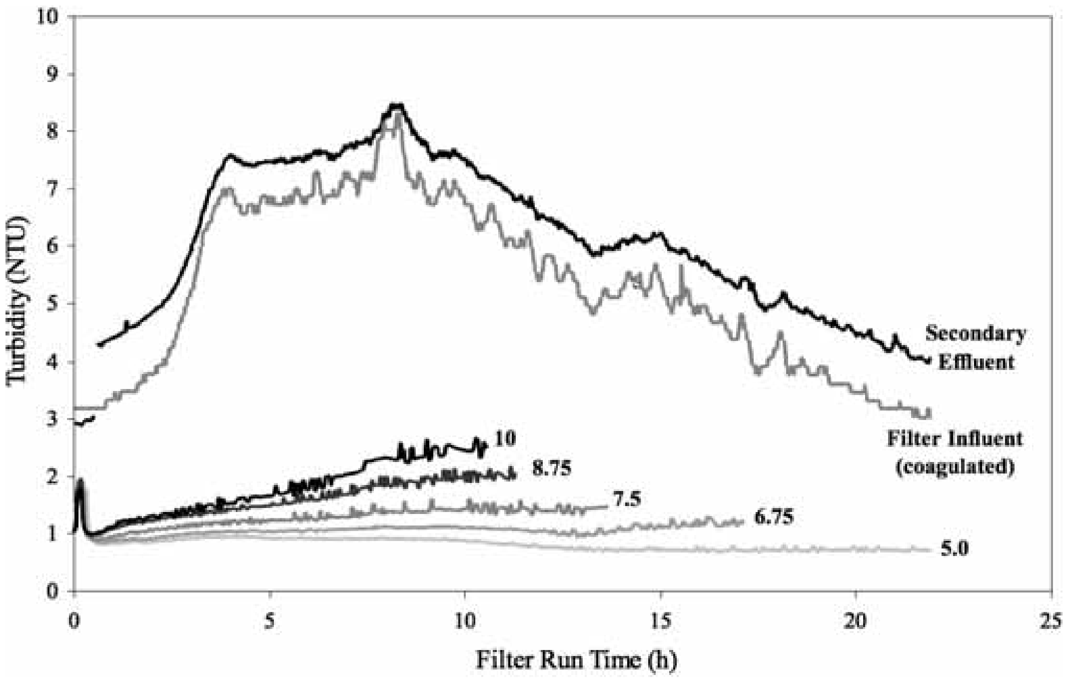


Figure 3.2. Secondary effluent (precoagulated), filter influent and effluent turbidity as a function of filter run time during an example filter run for loading rates 5–10 gal/ft^2-min.

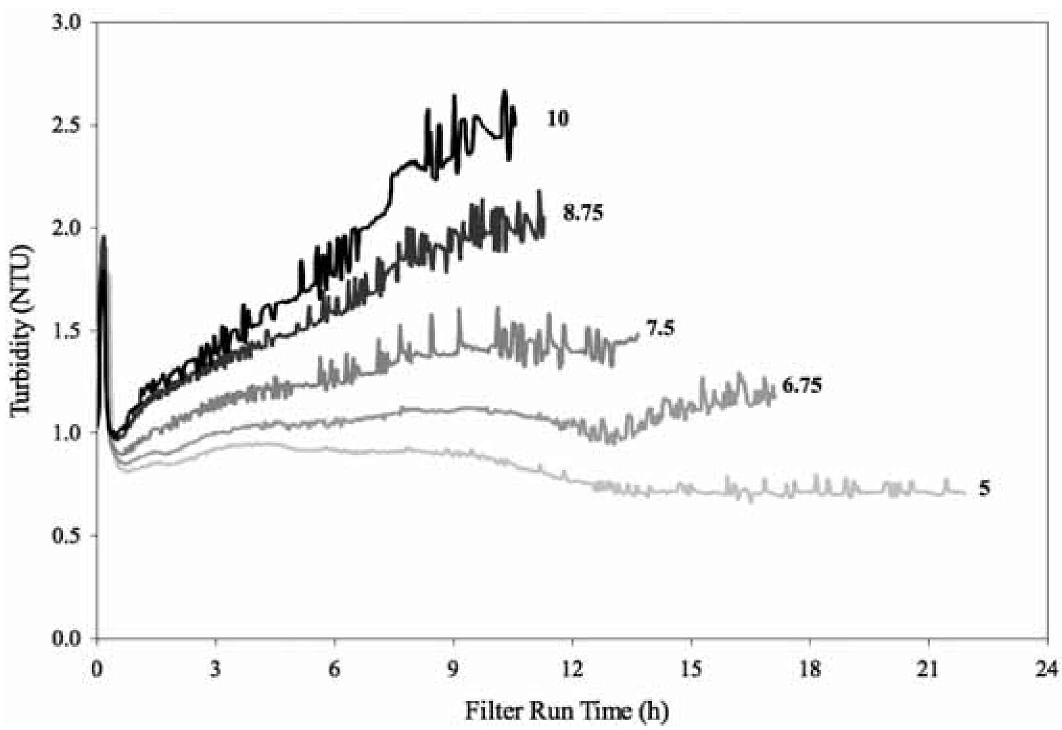


Figure 3.3. Filter effluent turbidity as a function of filter run time during an example filter run for loading rates 5–10 gal/ft^2-min (same data as Figure 3.2).

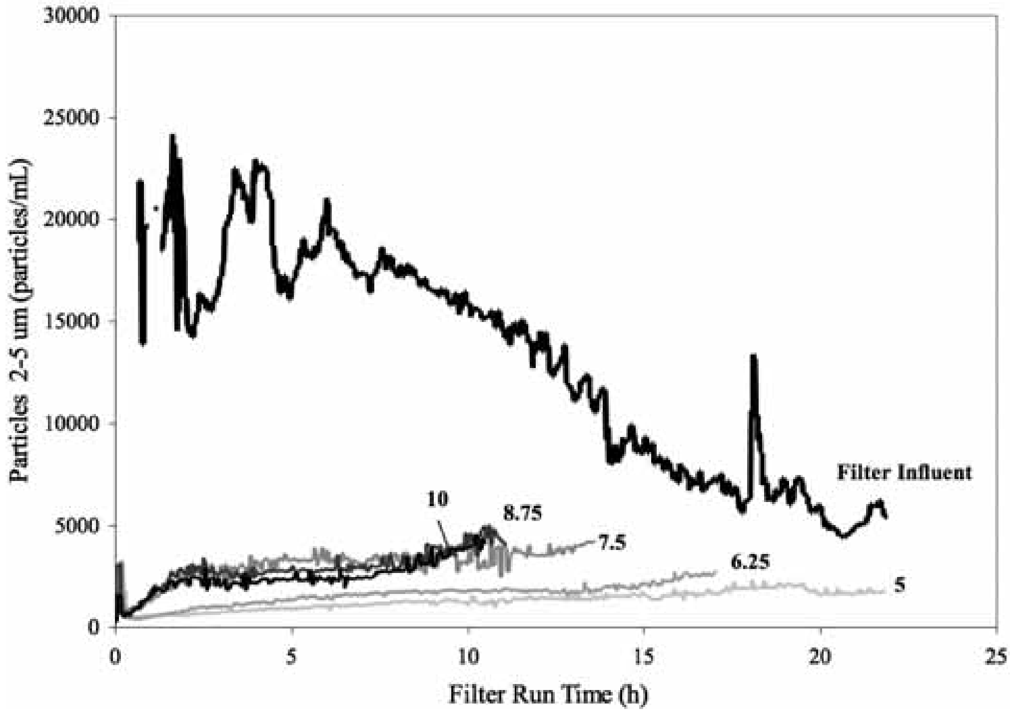


Figure 3.4. Filter influent and effluent particle counts in the 2–5 μm size range as a function of filter run time during an example filter run for loading rates 5–10 gal/ft^2-min.

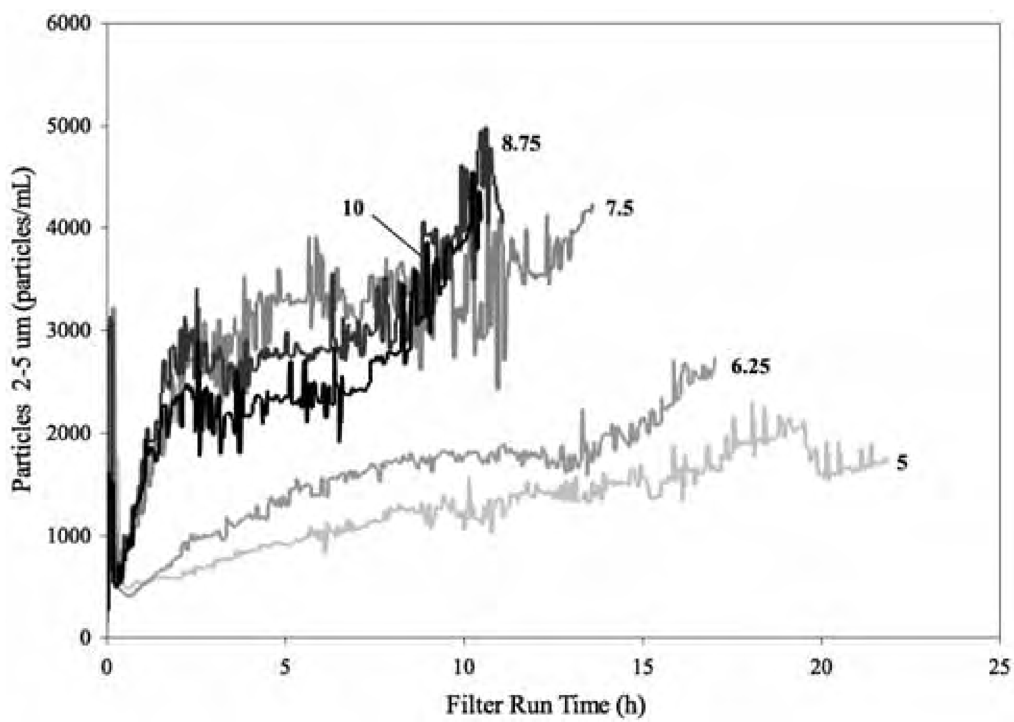


Figure 3.5. Filter effluent particle counts in the 2–5 μm size range as a function of filter run time during an example filter run for loading rates 5–10 gal/ft^2-min (same data as Figure 3.4).

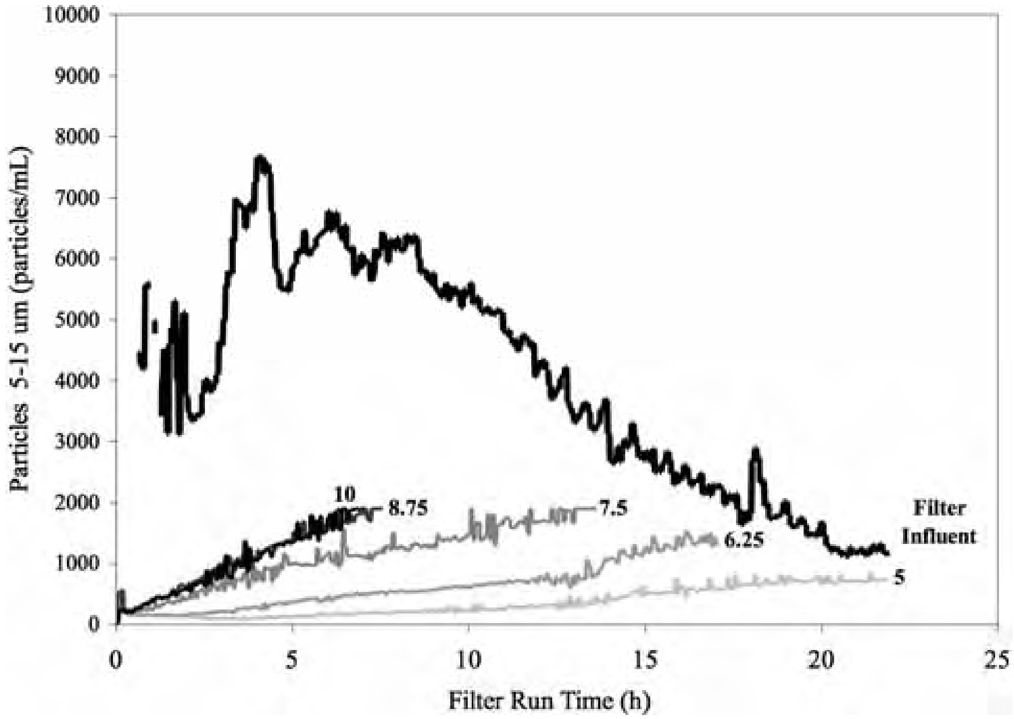


Figure 3.6. Filter influent and effluent particle counts in the 5–15 μm size range as a function of filter run time during an example filter run for loading rates 5–10 gal/ft²-min.

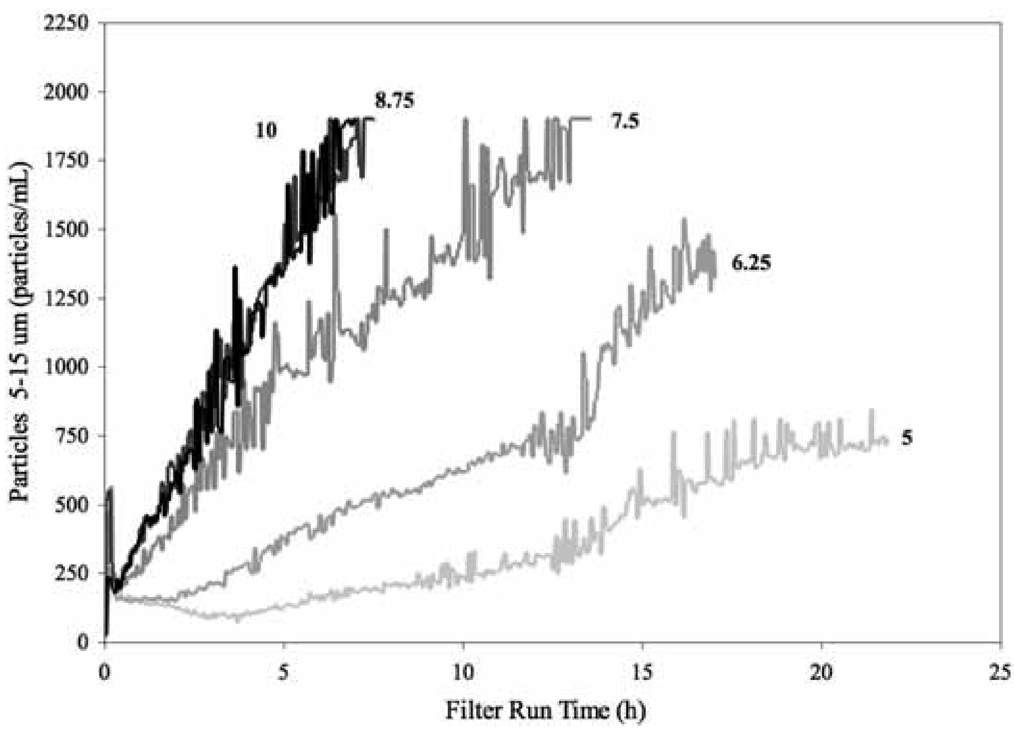


Figure 3.7. Filter effluent particle counts in the 5–15 μm size range as a function of filter run time during an example filter run for loading rates 5– 0 gal/ft²-min (same data as Figure 3.6).

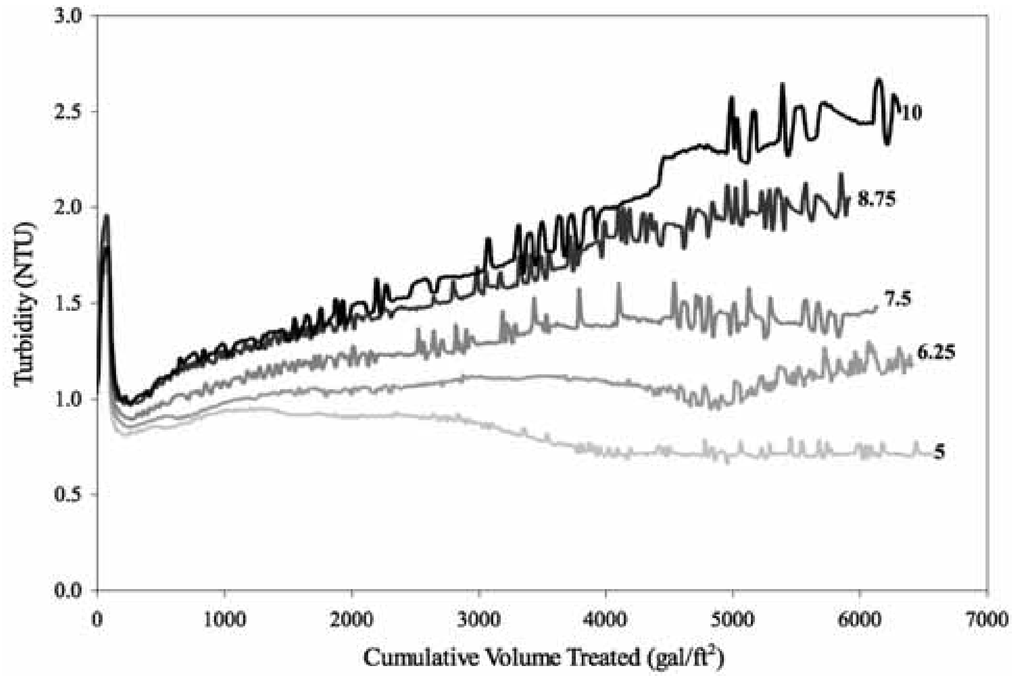


Figure 3.8. Filter effluent turbidity as a function of cumulative volume filtered during an example filter run for loading rates 5–10 gal/ft^2-min.

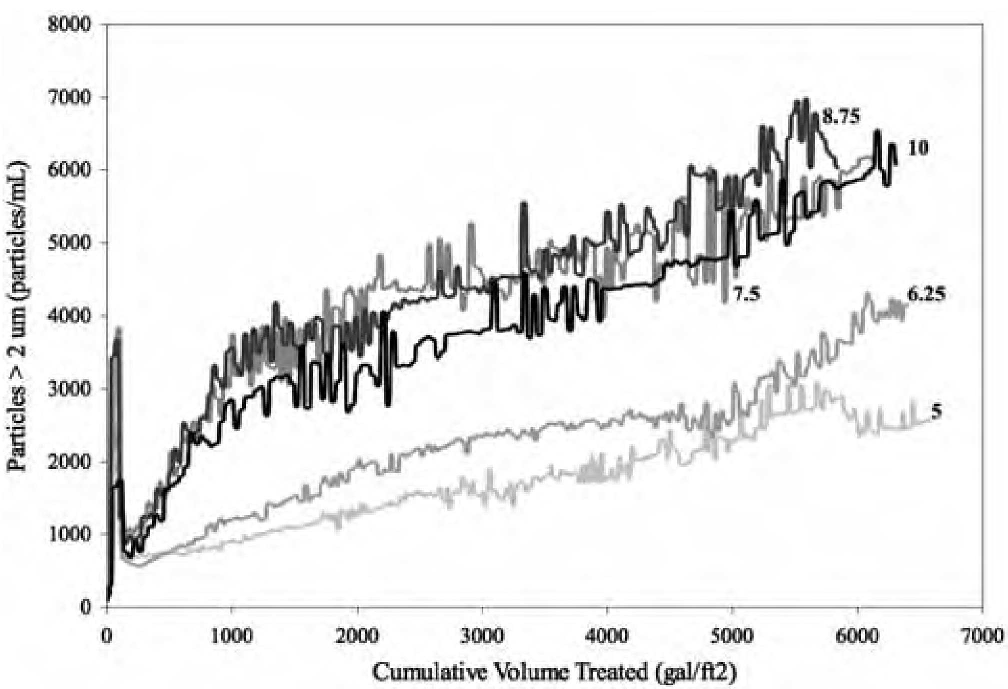


Figure 3.9. Filter effluent particle counts in the 2–5 μm size range as a function of cumulative volume filtered during an example filter run for loading rates 5–10 gal/ft^2-min.

In Figure 3.10, the average filter effluent particle counts in all seven size ranges are shown for one filter run, and in Figure 3.11 the corresponding log removals of particles are shown. As particle size increased, the number of particles decreased, but the removal efficiency was so variable that no trends with particle size are visible from a single filter run. A trend of decreasing removal efficiency with higher filter loading rate is visible, however.

In all of these plots, some variability is attributed to the particularities of each filter column and its analytical instruments. This variability highlights the importance of using the average performance from a set (typically 10) of filter runs at each loading rate for the data analysis. Nonetheless, the individual filter plots were viewed qualitatively for every filter run. These plots were extremely valuable for understanding filter performance and for verifying that no problems had occurred during each run.

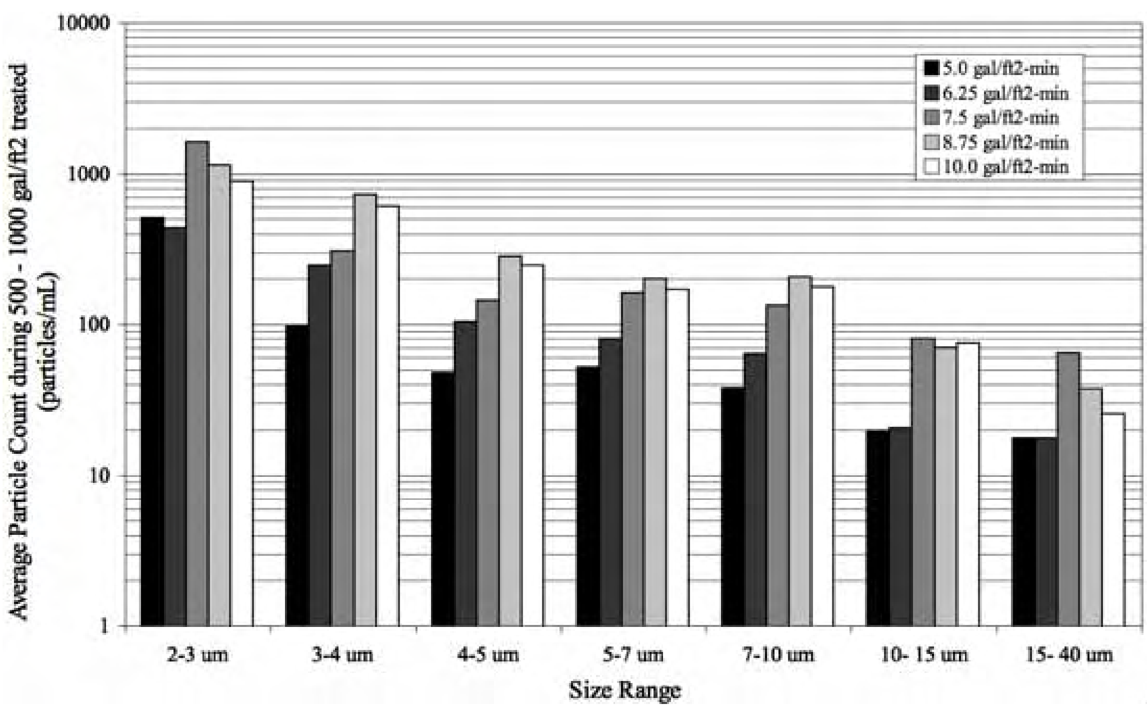


Figure 3.10. Average particle counts in the filter effluent as a function of particle size range during an example filter run for a given cumulative volume of water filtered (500 to 1000 gal/ft^2) for loading rates 5–10 gal/ft^2-min.

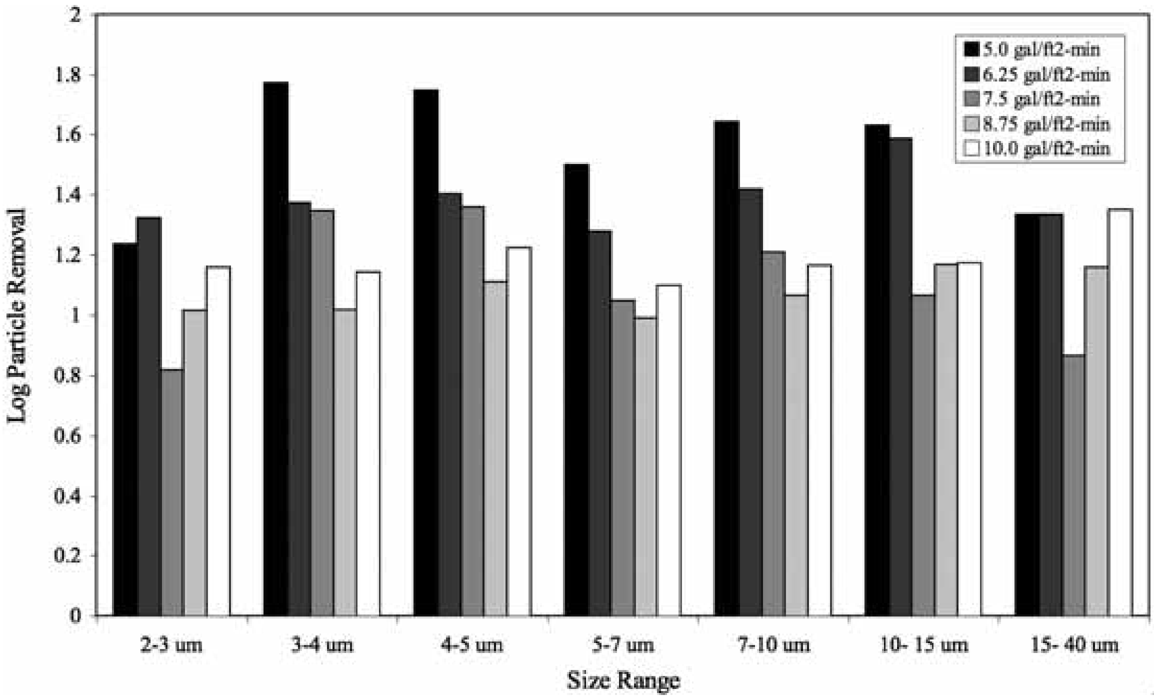


Figure 3.11. Average log removal of particles through filter as a function of particle size range during an example filter run for a given cumulative volume of water filtered (500 to 1000 gal/ft^2) for loading rates 5–10 gal/ft^2-min.

3.3 PILOT-FILTER PERFORMANCE WHEN ALL FILTER LOADING RATES RECEIVE SAME COAGULANT DOSE

3.3.1 Results From First Pilot Test Period (ISP1)

A summary of the operational characteristics from the first set of filter runs (ISP1) is provided in Table 3.2. The drop off in the volume of water treated at the 12.5 gal/ft^2-min is evident and is the primary reason this loading rate was dropped during subsequent test periods. The average influent and effluent turbidity and particle counts for the five loading rates are shown in Table 3.3. It is important to note that there were no significant differences in the average influent turbidity for any of the loading rates. (Note that influent particle counts were not measured until ISP3, so no values are provided.) There were no statistical differences between the average effluent turbidities at 5.0, 7.5, and 10.0 gal/ft^2-min, whereas the turbidity at 2.5 gal/ft^2-min was significantly lower than at 5.0 gal/ft^2-min, and the turbidity for the 12.5 gal/ft^2-min rate was higher. The effect of loading rate on effluent particle counts was similar to turbidity.

Table 3.2. Average[4] Filter Run Characteristics by Loading Rate for First Test Period (ISP1).

Loading Rate (gal/ft^2-min)	Coagulant Dose (mg/L)	Polymer Dose (mg/L)	Run Time (h)	Volume Treated (gal/ft^2)
2.5	14.5	1.0	29.8 ± 10.1	4697 ± 1300
5.0	14.5	1.0	20.8 ± 4.0	6414 ± 831
7.5	14.5	1.0	**13.3 ± 2.7**	6082 ± 1000
10.0	14.5	1.0	**9.7 ± 0.8**	5809 ± 479
12.5	14.5	1.0	**7.2 ± 0.9**	5364 ± 675

Note. Values given are mean + 95% confidence interval. Bolded values are significantly different from the 5 gal/ft^2-min value.

Table 3.3. Average[4] Turbidity and Particle Count Data for First Test Period (ISP1).

Loading Rate (gal/ft^2-min)	Turbidity (NTU)		Effluent Particle Counts (100 per mL)	
	Influent	Effluent	2-5 mm	5-15 mm
2.5	5.66 ± 0.32	**1.47 ± 0.20**	**27.9 ± 6.2**	**3.10 ± 0.68**
5.0	5.82 ± 0.41	2.00 ± 0.22	46.5 ± 6.8	10.7 ± 1.1
7.5	6.00 ± 0.52	2.25 ± 0.13	46.1 ± 7.3	11.8 ± 1.4
10.0	5.93 ± 0.49	2.30 ± 0.15	47.3 ± 6.7	12.8 ± 1.2
12.5	5.83 ± 0.51	**2.33 ± 0.10**	52.1 ± 5.0	**14.3 ± 0.75**

Note. Values given are mean + 95% confidence interval. Bolded values are significantly different from the 5 gal/ft^2-min value.

The average log removals of total coliform bacteria, *E. coli*, and MS2 coliphage are reported in Table 3.4A. Removals were slightly greater than 0.5 log, and no differences between loading rates were observed with the exception of *E. coli* at 12.5 gal/ft^2-min. The removals of indicator organisms that were due to filtration alone (not including coagulation) were also calculated, and are reported in Table 3.4B. It is interesting that most of the removal of the bacteria occurred during filtration, whereas very little removal of MS2 occurred during filtration. Thus, an "apparent" removal of MS2 occurred because of coagulation alone, which could have been due either to inactivation, or to aggregation of MS2 (one floc containing multiple MS2 phage would enumerate as a single plaque, resulting in an apparent decrease in MS2 concentration).

[4] Average while filter effluent turbidity was less than 2.49 NTU (California Water Recycling Criteria Limit). Of the 11 filter runs per rate, 0, 1, 2, 3, and 3 runs ended early because of turbidity for loading rates of 2.5, 5, 7.5, 10, and 12.5 gal/ft2-min respectively.

Table 3.4a. Average Log Removals of Microbiological Parameters Through Coagulation/Flocculation and Filtration During First Test Period (ISP1)

Loading Rate (gal/ft^2-min)	Total Coliforms	*E. coli*	MS2
2.5	0.66 ± 0.09	0.56 ± 0.09	0.72 ± 0.23
5.0	0.68 ± 0.08	0.62 ± 0.10	0.64 ± 0.21
7.5	0.67 ± 0.08	0.57 ± 0.09	0.75 ± 0.21
10.0	0.65 ± 0.10	0.53 ± 0.08	0.61 ± 0.17
12.5	0.57 ± 0.07	***0.43 ± 0.07***	0.65 ± 0.18

Note. Bolded values are significantly different from the 5 gal/ft^2-min value.

Table 3.4b. Average Log Removals of Microbiological Parameters Through Filtration During First Test Period (ISP1)

Loading Rate (gal/ft^2-min)	Total Coliforms	*E. coli*	MS2
2.5	0.60 ± 0.08	0.43 ± 0.08	0.15 ± 0.14
5.0	0.62 ± 0.08	0.48 ± 0.09	0.09 ± 0.13
7.5	0.60 ± 0.08	0.44 ± 0.08	0.21 ± 0.12
10.0	0.59 ± 0.09	0.40 ± 0.07	0.06 ± 0.08
12.5	0.53 ± 0.10	**0.29 ± 0.07**	0.10 ± 0.11

Note. Bolded values are significantly different from the 5 gal/ft^2-min value.

3.3.2 Results From Second Pilot Test Period (ISP2)

A summary of the operational characteristics from the second set of filter runs (ISP2) is provided in Table 3.5. All loading rates produced similar volumes of water per filter run. In Table 3.6 and Figure 3.13, the average influent and effluent turbidity and particle counts in the 2–5 and 5–15 μm size ranges are shown for the five loading rates. The influent quality was statistically equivalent for all loading rates, except for 5–15 μm particles, which were lower in the influent for the 7.5, 8.75, and 10.0 gal/ft^2-min rates. The effluent quality decreased with an increase in filter loading rates, with significantly higher turbidity observed from rates of 7.5 gal/ft^2-min and higher, and particle counts higher at the 10.0 gal/ft^2-min rate.

Table 3.5. Average[5] Filter Run Characteristics by Loading Rate for Second Test Period (ISP2)

Loading Rate (gal/ft^2-min)	Coagulant Blend Dose (mg/L)	Run Time (h)	Volume Treated (gal/ft^2)
5.0	24.2	15.8 ± 1.0	4756 ± 309
6.25	24.2	13.1 ± 1.7	4894 ± 652
7.5	24.2	**10.5 ± 1.3**	4732 ± 568
8.75	24.2	**8.7 ± 1.1**	4562 ± 585
10.0	24.2	**7.6 ± 1.2**	4553 ± 715

Note. Values given are mean + 95% confidence interval. Bolded values are significantly different from the 5 gal/ft^2-min value.

Table 3.6. Average[5] Turbidity and Particle Count Data for Second Test Period (ISP2).

Loading Rate (gal/ft^2-min)	Turbidity (NTU)		2–5 μm particles (1000 per mL)[6]		5–15 μm particles (100 per mL)[6]	
	Influent	Effluent	Influent	Effluent	Influent	Effluent
5.0	7.30 ± 0.57	1.40 ± 0.35	33.4 ± 3.8	3.1 ± 1.2	97.8 ± 12.4	9.6 ± 1.3
6.25	7.27 ± 0.54	1.93 ± 0.31	31.6 ± 4.1	3.9 ± 0.9	77.7 ± 8.8	9.6 ± 1.2
7.5	7.08 ± 0.53	**2.10 ± 0.19**	30.8 ± 4.5	4.8 ± 1.2	**69.9 ± 9.3**	10.9 ± 1.8
8.75	6.94 ± 0.45	**2.22 ± 0.17**	31.0 ± 4.7	5.4 ± 1.2	**69.1 ± 8.4**	12.9 ± 2.1
10.0	6.64 ± 0.52	**2.30 ± 0.14**	31.3 ± 5.0	***5.8 ± 1.0***	**70.5 ± 9.4**	**13.6 ± 1.8**

Note. Values given are mean + 95% confidence interval. Bolded values are significantly different from the 5 gal/ft^2-min value.

[5] To meet the California Water Recycling Criteria turbidity limit, of the 10 filter runs per loading rate, for the analysis, 0, 2, 2, 5, and 4 runs were ended prior to terminal head loss, when the average turbidity reached 2.49 NTU, for loading rates of 5, 6.25, 7.5, 8.75, and 10 gal/ft^2-min respectively.
[6] Because of particle concentrations in the filter effluent exceeding the instrument maximum concentrations, these particle count values are only for the time period while the filter effluent particle count was less than 2.0 NTU.

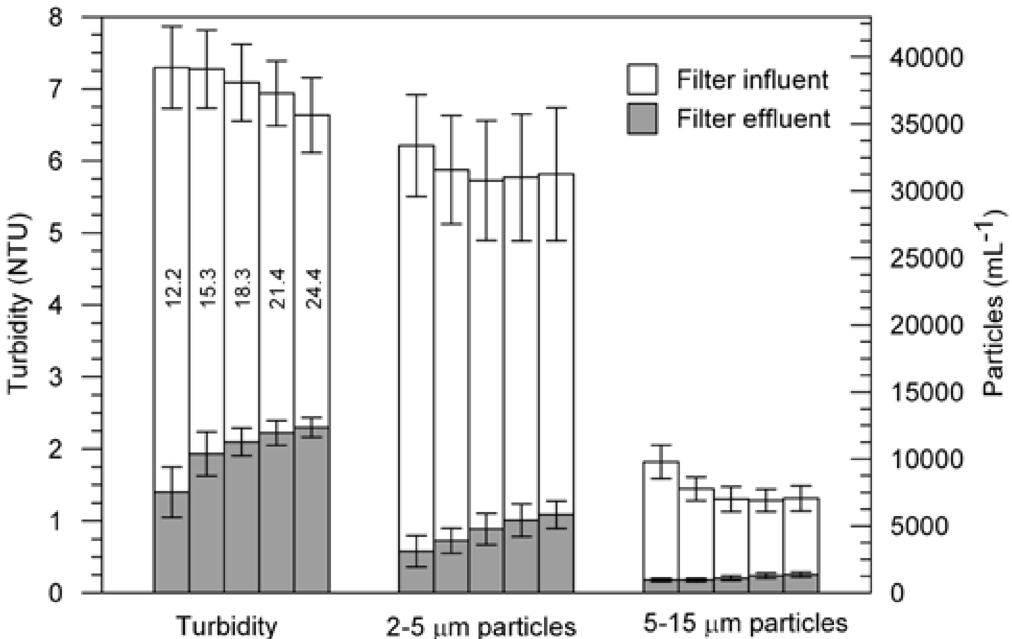


Figure 3.13. Impact of filter loading rate (indicated on bar, gal/ft^2-min) on turbidity and particle counts during the second test period (ISP2). Error bars indicate 95% confidence interval on mean.

The average log removals of indicator organisms across coagulation and filtration, and for just filtration, are shown in Tables 3.7a and 3.7b, respectively. A trend of lower removal as filter loading rate increased was observed, although significantly lower removal was only observed for *E. coli* at the highest loading rates. Comparing the values in the tables, it is seen that greater than one log of "apparent" removal of MS2 occurred during coagulation, as well as about 0.5 log removal of *E. coli*.

Table 3.7a. Average Log Removals of Microbiological Parameters Through Coagulation/Flocculation and Filtration During Second Test Period (ISP2)

Loading Rate (gal/ft^2-min)	Total Coliforms			*E. coli*			MS2		
5.0	1.20	±	0.17	1.50	±	0.13	3.00	±	0.25
6.25	1.05	±	0.21	1.34	±	0.11	2.51	±	0.24
7.5	1.01	±	0.16	1.29	±	0.10	2.53	±	0.18
8.75	0.98	±	0.18	**1.25**	**±**	**0.09**	2.43	±	0.22
10.0	0.92	±	0.15	**1.17**	**±**	**0.10**	2.32	±	0.18

Note. Bolded values are significantly different from the 5 gal/ft^2-min value.

Table 3.7b. Average Log Removals of Microbiological Parameters Through Filtration During Second Test Period (ISP2)

Loading Rate (gal/ft^2-min)	Total Coliforms			*E. coli*			MS2		
5.0	1.02	±	0.17	0.80	±	0.17	1.34	±	0.20
6.25	0.90	±	0.20	0.63	±	0.15	0.89	±	0.20
7.5	0.83	±	0.18	0.59	±	0.12	0.89	±	0.15
8.75	0.86	±	0.17	0.55	±	0.13	0.79	±	0.21
10.0	0.76	±	0.17	**0.47**	**±**	**0.14**	0.69	±	0.16

Note. Bolded values are significantly different from the 5 gal/ft^2-min value.

3.3.3 Results From Third Pilot Test Period (ISP3)

A summary of the operational characteristics from the third set of filter runs (ISP3) is provided in Table 3.8. All loading rates produced similar volumes of water per filter run. In Table 3.9 and Figure 3.14, the average influent and effluent turbidity and particle counts in the 2–5 and 5–15 μm size ranges are shown for the five loading rates. The influent quality was statistically equivalent for all loading rates. A trend of increasing turbidity and particle counts was observed as filter loading rate increased, with statistically significant higher values observed for 8.75 and 10.0 gal/ft^2-min (all parameters), 7.5 gal/ft^2-min (turbidity and 5–15 μm particles), and 6.25 gal/ft^2-min (5–15 μm particles).

The average log removals of indicator organisms across coagulation and filtration, and for just filtration, are shown in Tables 3.10a and 3.10b, respectively. A trend of lower removal as filter loading rate increased was observed, although significantly lower removal was only observed for *E. coli* at the highest loading rates. Comparing the values in the tables, it is seen that greater than one log of "apparent" removal of MS2 occurred during coagulation, whereas the removal of total coliform bacteria and *E. coli* by coagulation was very low.

Table 3.8. Average[7] Filter Run Characteristics by Loading Rate for Third Test Period (ISP3)

Loading Rate (gal/ft^2-min)	Coagulant Blend Dose (mg/L)	Run Time (h)	Volume Treated (gal/ft^2)
5.0	11.0	22.2 ± 2.0	6669 ± 613
6.25	11.0	**18.0 ± 1.3**	6743 ± 483
7.5	11.0	**14.8 ± 1.4**	6637 ± 635
8.75	11.0	**12.5 ± 1.0**	6566 ± 506
10.0	11.0	**11.0 ± 1.2**	6602 ± 723

Note. Values given are mean + 95% confidence interval. Bolded values are significantly different from the 5 gal/ft^2-min value.

Table 3.9. Average[7] Turbidity and Particle Count Data for Third Test Period (ISP3).

Loading Rate (gal/ft^2-min)	Turbidity (NTU)		2–5 mm particles (1000 per mL)		5–15 mm particles (100 per mL)	
	Influent	Effluent	Influent	Effluent	Influent	Effluent
5.0	6.50 ± 0.44	1.06 ± 0.16	21.3 ± 6.0	2.3 ± 0.8	76 ± 21	4.4 ± 1.5
6.25	6.68 ± 0.44	1.35 ± 0.18	22.0 ± 5.6	3.2 ± 1.0	79 ± 20	**9.7 ± 2.2**
7.5	6.59 ± 0.47	**1.60 ± 0.20**	22.6 ± 6.4	3.8 ± 0.9	77 ± 17	**12.2 ± 1.5**
8.75	6.46 ± 0.55	**1.81 ± 0.22**	22.6 ± 6.8	**5.1 ± 1.4**	73 ± 16	**13.9 ± 1.0**
10.0	6.30 ± 0.65	**1.94 ± 0.23**	22.9 ± 8.1	**5.8 ± 1.3**	71 ± 16	**15.1 ± 1.3**

Note. Values given are mean + 95% confidence interval. Bolded values are significantly different from the 5 gal/ft^2-min value.

[7] All filter runs (12 per rate) at all rates met turbidity requirements specified by the California Water Recycling Criteria Limit, and all runs were terminated because of reaching maximum head loss.

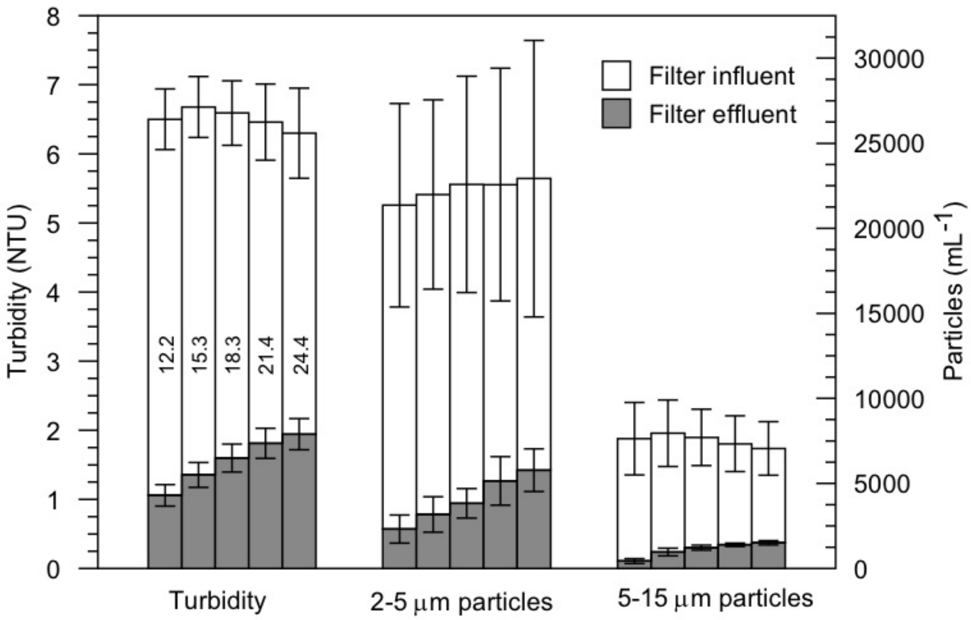


Figure 3.14. Impact of filter loading rate (indicated on bar, gal/ft^2-min) on turbidity and particle counts during the third test period (ISP3). Error bars indicate 95% confidence interval on mean.

Table 3.10a. Average Log Removals of Microbiological Parameters Through Coagulation/Flocculation and Filtration During Third Test Period (ISP3).

Loading Rate (gal/ft^2-min)	Total Coliforms	*E. coli*	MS2
5.0	0.91 ± 0.07	0.92 ± 0.07	2.37 ± 0.11
6.25	0.86 ± 0.06	0.86 ± 0.07	2.29 ± 0.11
7.5	0.78 ± 0.06	0.77 ± 0.06	2.18 ± 0.16
8.75	0.73 ± 0.07	***0.78 ± 0.08***	2.21 ± 0.11
10.0	0.68 ± 0.08	***0.70 ± 0.06***	2.20 ± 0.11

Note. Bolded values are significantly different from the 5 gal/ft^2-min value.

Table 3.10b. Average Log Removals of Microbiological Parameters Through Filtration During Third Test Period (ISP3).

Loading Rate (gal/ft^2-min)	Total Coliforms	*E. coli*	MS2
5.0	0.72 ± 0.07	0.66 ± 0.05	0.78 ± 0.09
6.25	0.66 ± 0.05	0.60 ± 0.05	0.70 ± 0.08
7.5	0.59 ± 0.06	0.50 ± 0.05	0.60 ± 0.16
8.75	0.53 ± 0.06	***0.51 ± 0.06***	0.62 ± 0.10
10.0	0.49 ± 0.07	***0.43 ± 0.06***	0.61 ± 0.10

Note. Bolded values are significantly different from the 5 gal/ft^2-min value.

3.3.4 Comparisons of Removal Efficiency During the First Three Pilot Test Periods

The removal efficiency of the pilot filters for the first three test periods is compared in this section. The data presented in these comparison plots are roughly the same as those from Sections 3.3.1 through 3.3.2. However, the definition of a filter run was modified slightly to overcome some differences between the three test periods. Thus, for each filter run, data points were averaged for the period until the filter effluent turbidity reached 2.0 NTU. This small difference in data analysis does not change the overall observed trends between test periods, but the values are slightly different from those in the previous tables.

In Figures 3.15 through 3.19, the treatment efficiency for each parameter (turbidity, particle counts, and indicator organisms) as a function of filter loading rate is compared for the three test periods. The effluent turbidity (Figure 3.15) varied considerably between the three ISPs, even though the average influent turbidity was fairly similar (ranging from 5 to 7 NTU). The main factor influencing the effluent turbidity was likely the polymer and coagulant dose. During ISP1, the dose of cationic polymer was much lower than in ISP2 and ISP3, when the combined polymer/coagulant (JC1679) was used. Similarly, the effluent turbidity during ISP2 was likely lower than ISP3 because of the higher coagulant dose used. A review of the summary tables for ISP1, ISP2, and ISP3 reveals that similar trends were observed with the other parameters. Thus, as expected, proper coagulation was essential to achieving adequate filtration. This effect was especially strong for removal of MS2 coliphage. It should also be noted that other water quality factors besides turbidity can have a significant impact on coagulation and filtration efficiency, and seasonal differences were experienced at the treatment plant.

The effluent turbidity increased as a function of loading rate during all three ISPs (Figure 3.15). However, the effect of loading rate was smaller during ISP1, which may also be a result of the lower polymer dose used. Unfortunately it is not possible to independently understand the impact of polymer versus coagulant dose, because the two were not varied independently in the study. The interaction of coagulant dose and loading rate is discussed further in Section 3.3.6.

The removal efficiencies of particles in the 2–5 and 5–15 μm size ranges were similar during ISP2 and ISP3, despite the higher ISP2 coagulant doses (Figures 3.16 and 3.17). (The removal efficiency during ISP1 could not be calculated because the influent particle counts were not measured.) The impact of loading rate was also similar between the ISPs, with removal efficiency decreasing with increasing loading rate. The removals of MS2, total coliform, and *E. coli* decreased as loading rate increased during ISP2 and ISP3, but the impact of loading rate was minimal during ISP1 (Figures 3.18, 3.19, and 3.20). The removals were highest during ISP2, followed by ISP3, and ISP1, which correlates with the average level of coagulant/polymer added.

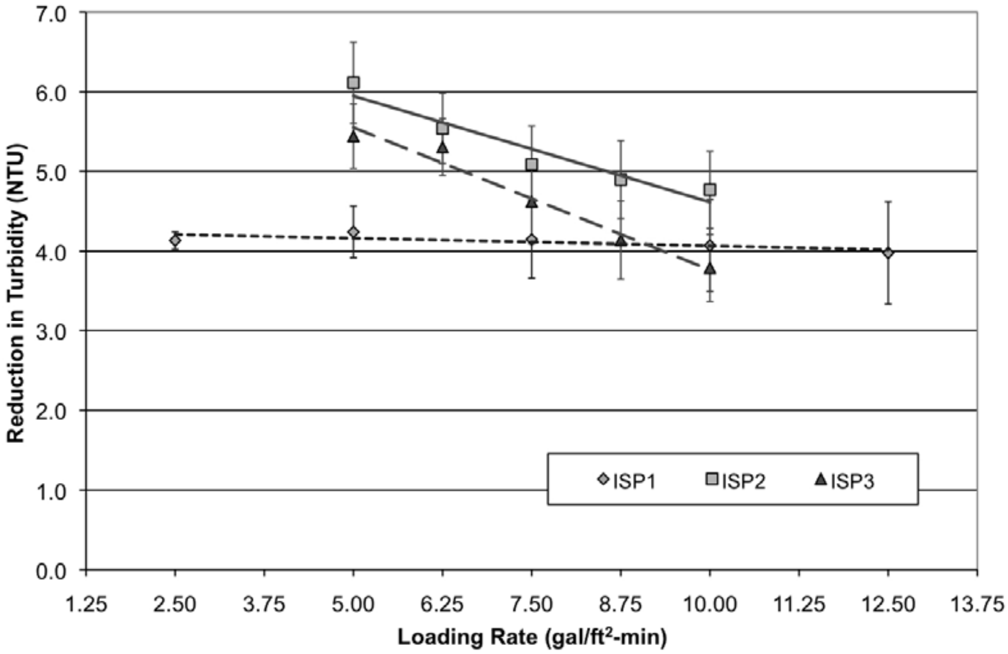


Figure 3.15. Comparison of filter effluent turbidity (while filter effluent turbidity was less than 2.0 NTU) among the three ISP testing periods.

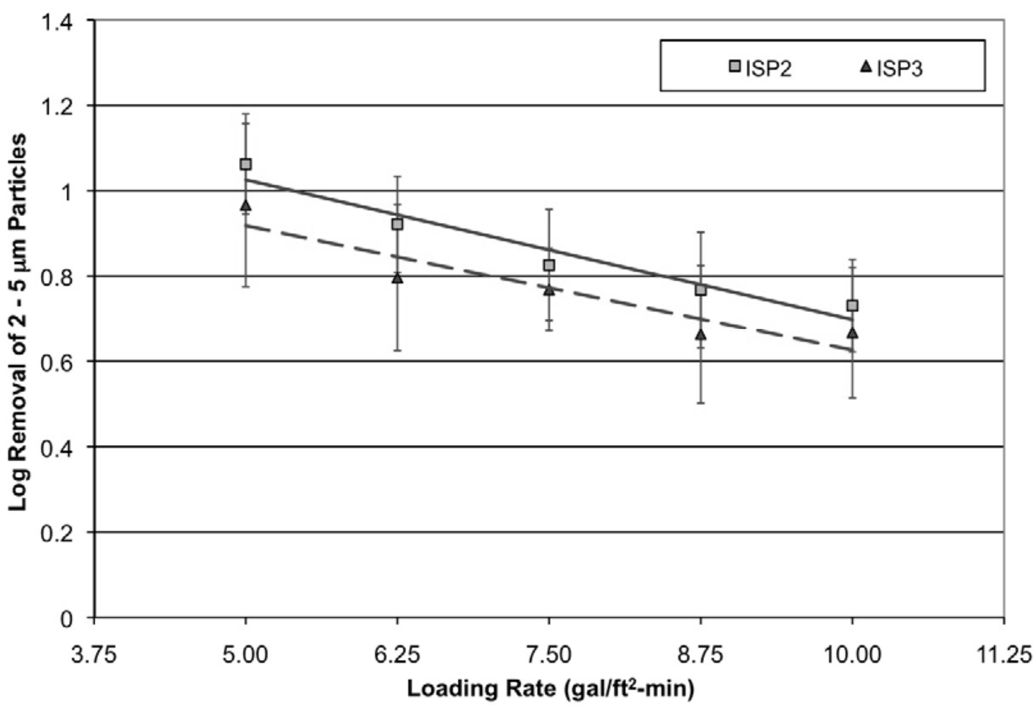


Figure 3.16. Comparison between two ISP testing periods of log 2–5 μm particle removal by loading rate (while filter effluent turbidity was less than 2.0 NTU).

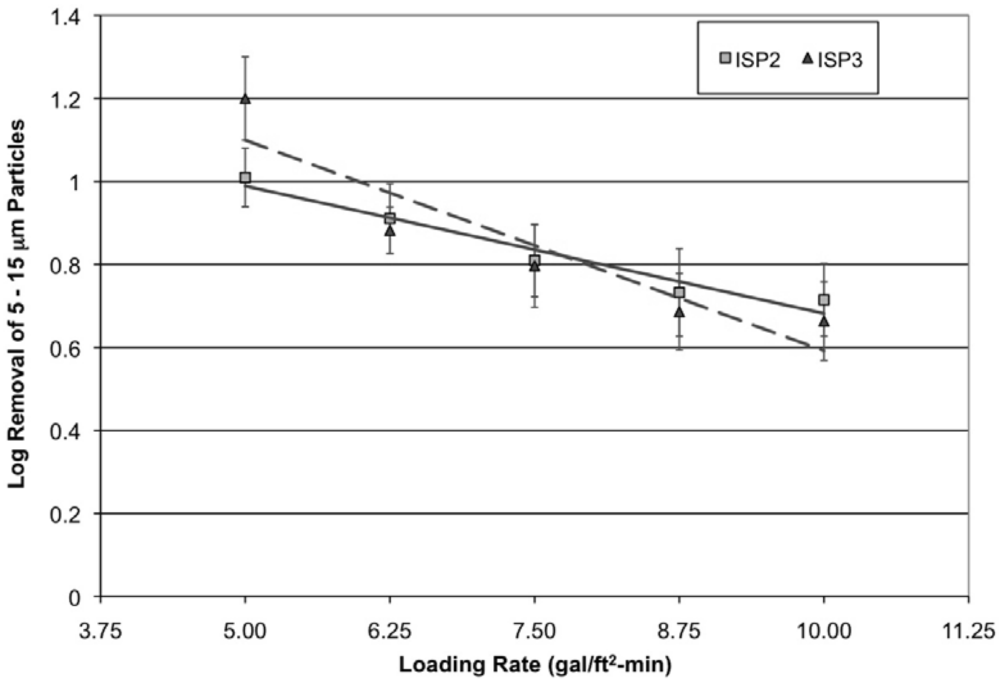


Figure 3.17. Comparison between two ISP testing periods of log 5–15 μm particle removal by loading rate (while filter effluent turbidity was less than 2.0 NTU).

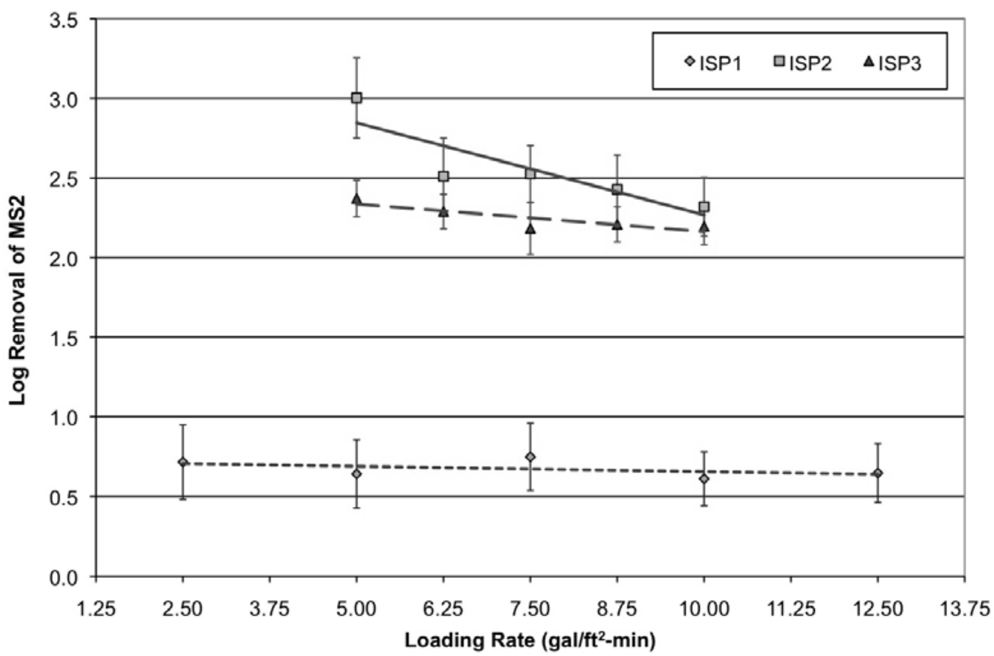


Figure 3.18. Comparison of log removal of MS2 through coagulation and filtration versus loading rate among the three ISP testing periods.

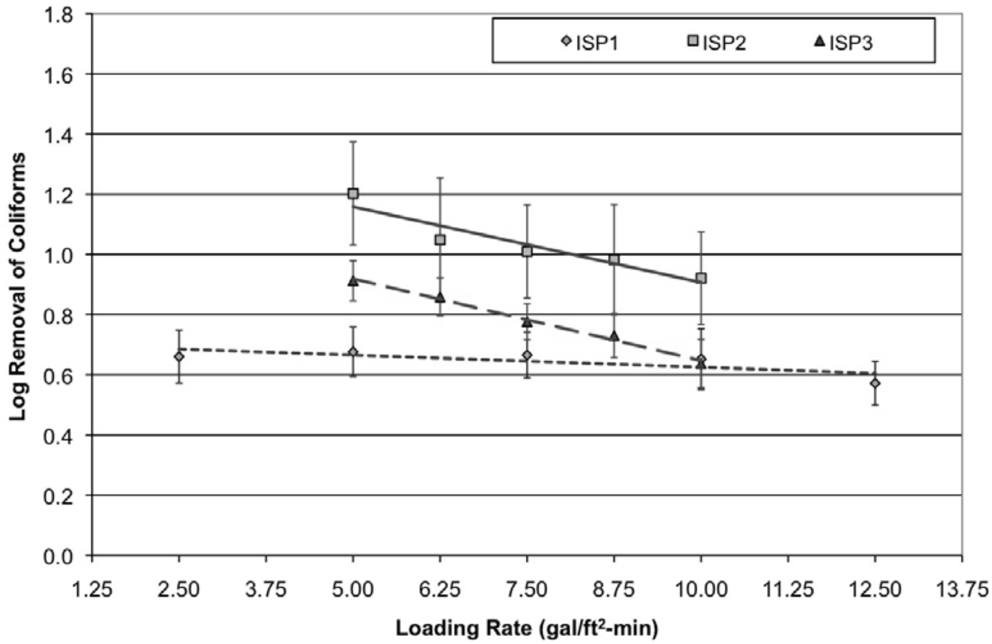


Figure 3.19. Comparison of log removal of total coliform bacteria through coagulation and filtration versus loading rate among the three ISP testing periods.

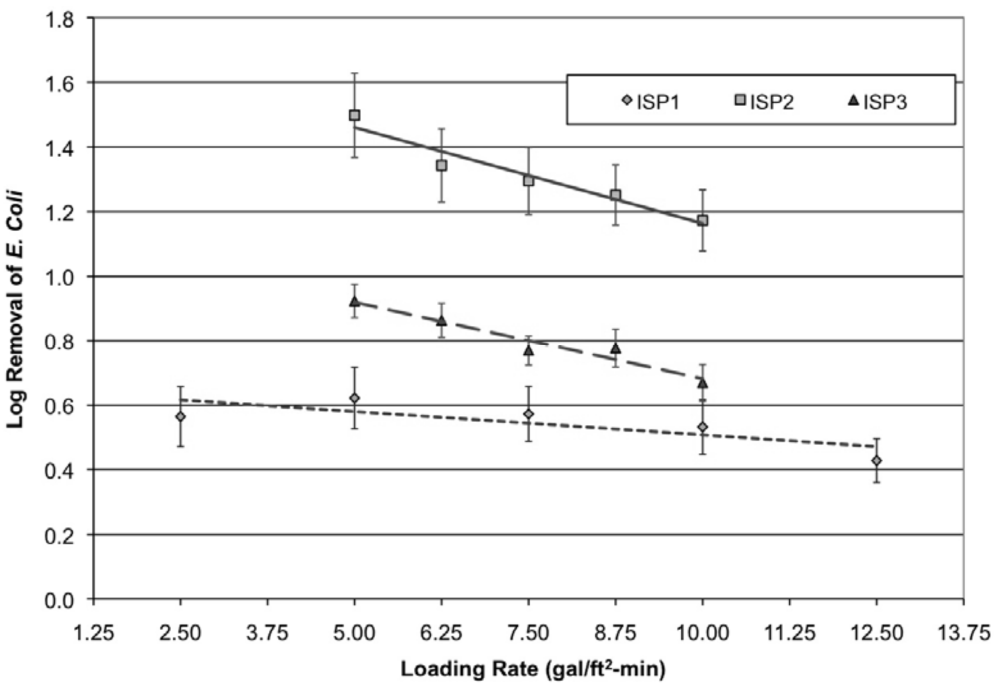


Figure 3.20. Comparison of log removal of *E. coli* through coagulation and filtration versus loading rate among the three ISP testing periods.

3.3.5 Comparison of Initial Removal Efficiency With Clean Bed Filtration Theory

The negative impact of loading rate on filtration removal efficiency observed at the pilot plant was expected based on clean bed filtration theory. However, many factors may influence how strong the effect of loading rate is; thus, it is not possible to predict performance from filtration models. In this section, the observed removals at the pilot plant are compared with theoretical clean bed filtration models.

The clean bed particle removal by the pilot filters was defined as the removal during the first 5 min of operation. Three different theoretical models were used for comparison (Rajagopalan and Tien, 1976; Tufenkji and Elimelech, 2004; Yao et al., 1971). Values for the filtration model variables were determined from direct measurements in the filter beds, when possible, and using literature values. In all cases the attachment efficiency (α) was arbitrarily set equal to one.

In Figure 3.21, the measured and predicted removal efficiencies as a function of particle diameter at a filter loading rate of 5 gal/ft^2-min are shown for the ISP2, ISP3, and SRO test periods. A similar plot for the 7.5 gal/ft^2-min rate is shown in Figure 3.22. Consistent with theory, the smaller particles were removed less effectively than larger particles by the pilot filters (in the 2–15 μm size range). Although the magnitude of the removals did not agree with theory, it is a significant finding that the predicted impact of particle size on removal efficiency could be detected in pilot-scale filters treating actual wastewater, using online continuous particle counters.

Although particle count data were not available for smaller diameters, the removals of *E. coli* and MS2 are also shown for the three test periods, assuming nominal diameters of 1.7 and 0.023 μm, respectively (Calendar, 2006; Madigan et al., 2000). The removal trends for these two organisms were also roughly consistent with theory, in that *E. coli* was removed with lower efficiency than MS2 for two of the test periods. However, it should be noted that both *E. coli* and MS2 could be removed either as individual particles or as incorporated into larger particles that are subsequently removed. Coagulation can potentially improve removal by affecting both of these mechanisms, for example, by decreasing the net negative surface charge of the organisms and by increasing their incorporation into larger particles. It is not known how the surface characteristics of *E. coli* and MS2 compared under the conditions of these experiments, nor whether these organisms were associated with other particles, so comparing the observed removals with clean bed theory may not be justified.

Significantly better than predicted clean bed removal was observed for the smallest measured particles (2–3 μm), and the difference between predicted and observed values decreased with increasing particle size. Several factors may explain the better than predicted particle removal, including (1) particles remaining on the filter media after backwashing enhanced removal by providing additional and more effective collector surfaces, such that the media bed was not truly “clean”; (2) high particle concentrations in the wastewater may have caused the ripening to occur quickly, such that the observed removals are actually postripening; (3) straining may have been a significant mechanism of particle deposition (Tufenkji et al., 2004), one that is unaccounted for in the clean bed model; and (4) heterogeneity in the media that resulted in media size and porosity gradients (as a function of depth) during media stratification at the end of backwashing, which could result in better than expected removal at the top of the media. Further, there are several factors that could cause the difference between observed and predicted values to decrease with increasing particles size, including (1) flocculation in the filter may have created a shift in the particle size distribution, creating the appearance of greater removal of smaller particles and subsequently less removal of larger particles; (2) greater detachment of larger particles; and (3) heterogeneity in the particle densities with size.

Another way to visualize the impact of loading rate on particle removal efficiency is to plot removal efficiency versus loading rate, as illustrated in Figures 3.23 and 3.24 for two particle size ranges, 2–5 μm and 5–15 μm, respectively. Again, the observed trends that are due to an increase in loading rate are remarkably consistent with clean bed theory, although the differences between the magnitude of the observed and predicted clean bed removals, as discussed earlier, are also reflected here. There were no statistically significant differences in slopes between linear regression lines fit through the experimental and theoretical data.

The average removal of turbidity and particles for the entire filter runs are also plotted in Figures 3.23 and 3.24. The impact of loading rate was greater over the entire run than at the beginning of the run; this effect could also seen in Figures 3.3 through 3.7, as the difference between filter effluent turbidities increased throughout the run. The mean particle diameter (over 2–15 μm size range) in the filter effluent also increased throughout the filter runs for all loading rates (data not shown).

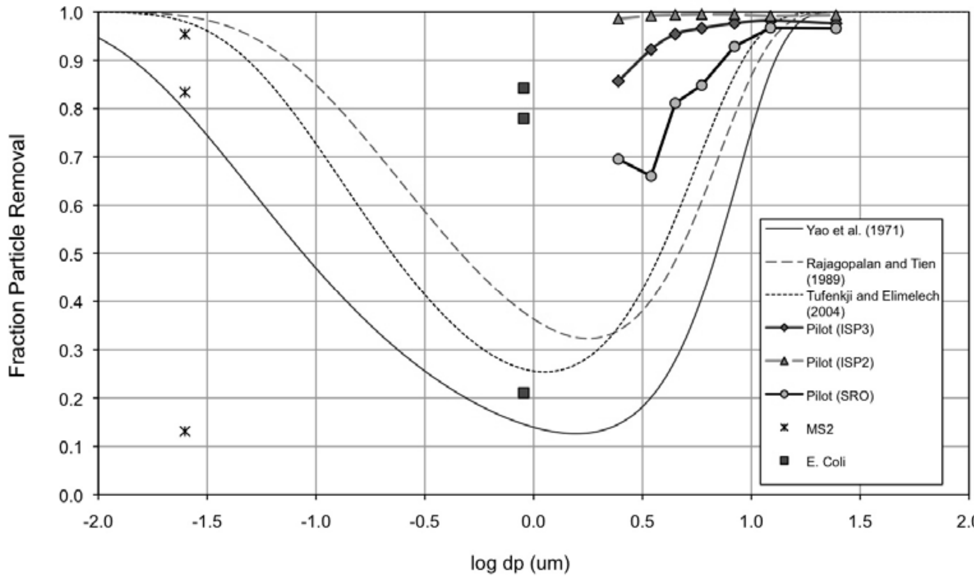


Figure 3.21. Comparison of initial removal through the pilot filters and predicted clean bed removal from theoretical models (assuming ideal conditions) for various particle size ranges at a loading rate of 5 gal/ft^2-min.

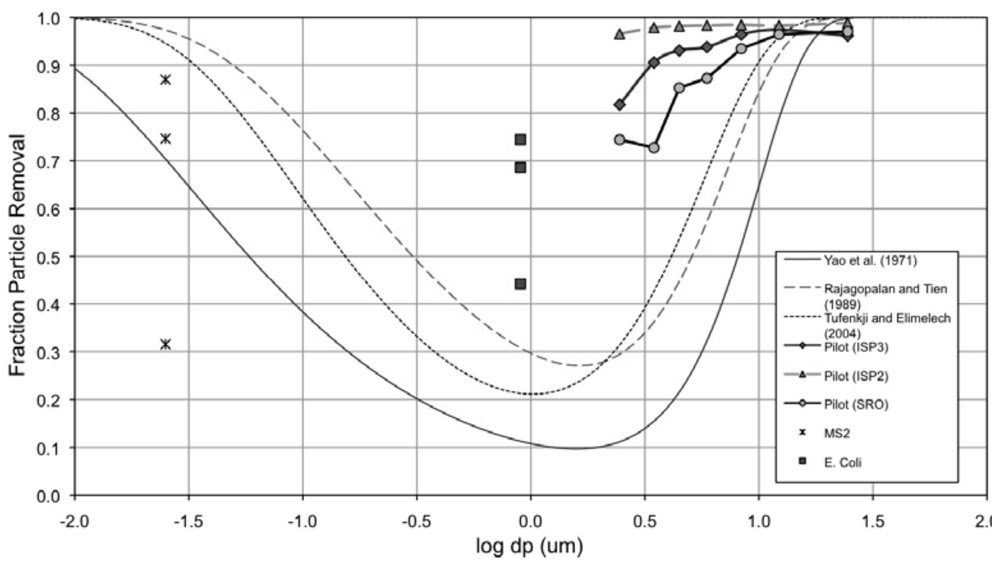


Figure 3.22. Comparison of initial removal through the pilot filters and predicted clean bed removal from theoretical models (assuming ideal conditions) for various particle size ranges at a loading rate of 7.5 gal/ft^2-min.

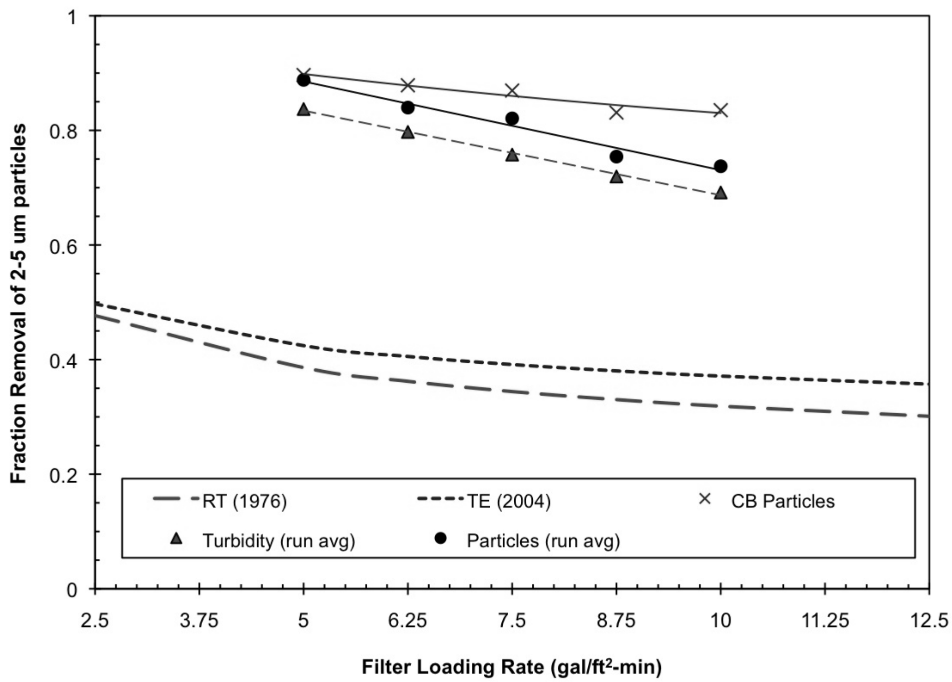


Figure 3.23. Comparison of theoretical clean bed removal with measured removal of 2–5 µm particles in the pilot filters as a function of loading rate (ISP3). The initial (CB) removal of particles was measured during the first 5 min of the filter run, whereas the run averages include data collected during the entire filter run.

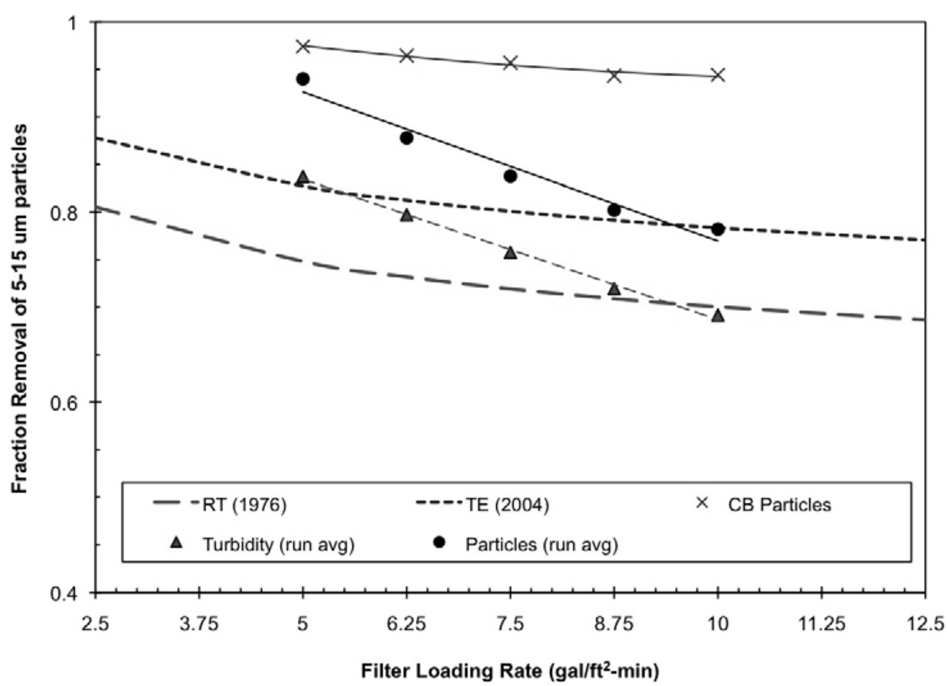


Figure 3.24. Comparison of theoretical clean bed removal with measured removal of 5–15 µm particles in the pilot filters as a function of loading rate (ISP3). The initial (CB) removal of particles was measured during the first 5 min of the filter run, whereas the run averages include data collected during the entire filter run.

3.3.6 Impact of Coagulant Dose on Removal Efficiency as a Function of Loading Rate

A comparison of the three ISP test periods led to the observation that the difference between loading rates seemed to be greater when a higher coagulant dose was used (which typically also resulted in a higher removal efficiency). To further illustrate this phenomenon, several additional filter runs were completed. In Figure 3.25, the results from a run comparing the five loading rates with no coagulant addition is shown. In this run, increasing the loading rate from 5.0 to 10.0 gal/ft^2-min only reduced the average fraction of turbidity removal over the entire run by 0.05 (from a removal fraction of 0.56 to 0.51). Another series of runs, shown in Figure 3.26, was conducted in which the coagulant dose was increased in a stepwise fashion throughout the run. When the coagulant dose increased, the difference in filter effluent turbidity between loading rates also increased (over the range of 0–12 mg/L of coagulant). The experimental setup used for this research did not allow the coagulant dose to vary for different filters, so the effect of coagulant dose and rate could not be independently studied. Thus, additional research is needed to better understand the interactions between coagulation and filtration, including particle formation, changes in the particle size distribution, and changes in particle surface characteristics.

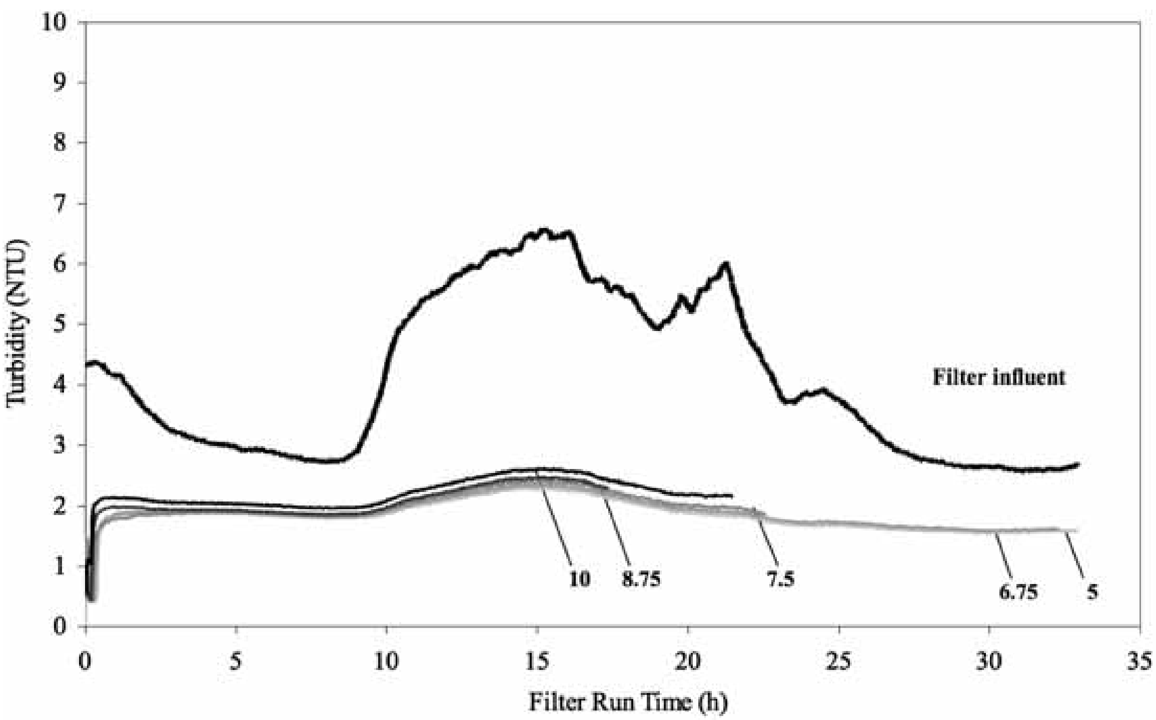


Figure 3.25. Filter effluent turbidity from filters operated at five loading rates when no coagulant or polymer are added prior to filtration (May 11–12, 2005; average turbidities ranged from 1.8 to 2.2 NTU, with higher turbidity with higher loading rates).

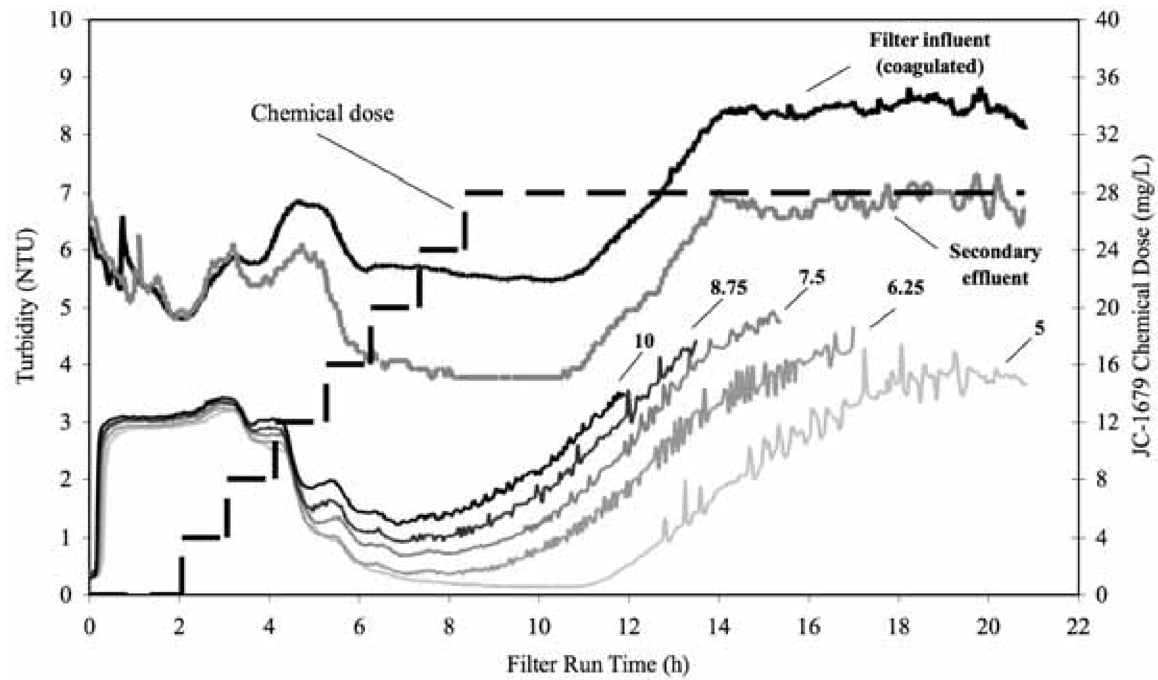


Figure 3.26. Impact of loading rate with increasing coagulant dose (April 27–28, 2005). Solid lines are turbidity values, shown on left y-axis; dashed line is the coagulant dose with values shown on the right y-axis.

3.4 PILOT FILTER PERFORMANCE WHEN OPTIMIZING COAGULANT DOSE SPECIFIC TO FILTER LOADING RATE

The objective of the fourth set of pilot tests was to determine if a higher coagulant dose could be used to overcome the lower removal efficiency associated with an increased filter loading rate. Two filter loading rates were tested, 5.0 and 7.5 gal/ft^2-min. The higher rate of 7.5 gal/ft^2-min was chosen based on the ISP test period results, which suggested that it was feasible from an operational perspective, and that the effluent quality was not far from that produced at 5.0 gal/ft^2-min. During the single rate optimization (SRO) runs, the coagulant dose was continuously optimized by the pilot plant operator such that the average effluent turbidity for each run was 1.90 NTU (corresponding to the treatment goal at the full-scale plant). An example of the coagulant dosing under these conditions was provided in Figure 2.10.

A summary of the operational characteristics from the SRO runs is provided in Table 3.11. The mean coagulant dose was 3.5 mg/L for the 5.0 gal/ft^2-min rate and 5.6 for the 7.5 gal/ft^2-min rate. Larger volumes of water were produced at the higher loading rate, but this is largely an artifact because all filter runs were terminated after 24 h. As shown in Table 3.12 and Figure 3.27, the two loading rates experienced similar filter influent turbidity. The average filter effluent turbidities were equivalent, at 1.90 and 1.86 NTU for the 5.0 and 7.5 gal/ft^2-min rates, respectively. As shown in Table 3.13 and Figure 3.28, the influent particle counts were very similar. The effluent particle count in the 2–5 μm size range was actually lower for the higher loading rate, whereas particle counts in the 5–15 μm range were statistically equivalent.

The average log removals of indicator organisms across coagulation and filtration, and for just filtration, are shown in Tables 3.14 and 3.15, respectively. The most interesting finding was that the removal of MS2 was significantly higher at the higher loading rate. Again, this finding is consistent with what was observed during the ISP runs and is attributed to the higher coagulant dose used. A comparison of the two tables reveals that the majority of the removal was attributed to the coagulation step. The removal of total coliform bacteria was similar for the two loading rates, whereas the removal of *E. coli* was slightly higher at the higher loading rate.

Table 3.11. Run Characteristics for Loading Rate Specific Coagulation Optimization Test Period (SRO).

Loading Rate (gal/ft^2-min)	Coagulant (mg/L)	Filter Run Time (h)	Volume Treated (gal/ft^2)
5.0	3.5 ± 2.2	24.0 ± 0.0	7211 ± 1
7.5	5.6 ± 0.4	22.7 ± 0.6	10200 ± 650

Table 3.12. Turbidity During Loading Rate Specific Coagulation Optimization Test Period (SRO).

Loading Rate	Turbidity (NTU)		
(gal/ft^2-min)	Secondary	Influent	Effluent
5.0	3.77 ± 0.42	5.03 ± 0.65	1.90 ± 0.04
7.5	3.62 ± 0.39	5.36 ± 0.61	1.86 ± 0.04

Table 3.13. Particle Counts During Loading Rate Specific Coagulation Optimization Test Period (SRO).

Loading Rate	2-5 μm Particles (1000 per mL)		5-15 μm Particles (100 per mL)	
(gal/ft^2-min)	Influent	Effluent	Influent	Effluent
5.0	14.8 ± 3.3	4.61 ± 0.20	57 ± 11	833 ± 47
7.5	14.7 ± 1.7	3.85 ± 0.12	59 ± 9	788 ± 43

Table 3.14. Log Removal of Microbial Parameters Through Coagulation and Filtration During Loading Rate Specific Coagulation Optimization Test Period (SRO).

Loading Rate (gal/ft^2-min)	Total Coliform (log removal)	*E. coli* (log removal)	MS2 (log removal)
5.0	0.28 ± 0.08	0.06 ± 0.05	0.25 ± 0.16
7.5	0.29 ± 0.04	0.25 ± 0.04	1.58 ± 0.11

Table 3.15. Log Removal of Microbial Parameters Through Filtration Only During Loading Rate Specific Coagulation Optimization Test Period (SRO).

Loading Rate (gal/ft^2-min)	Total Coliform (log removal)	*E. coli* (log removal)	MS2 (log removal)
5.0	0.25 ± 0.06	0.10 ± 0.04	0.06 ± 0.10
7.5	0.26 ± 0.04	0.25 ± 0.05	0.16 ± 0.14

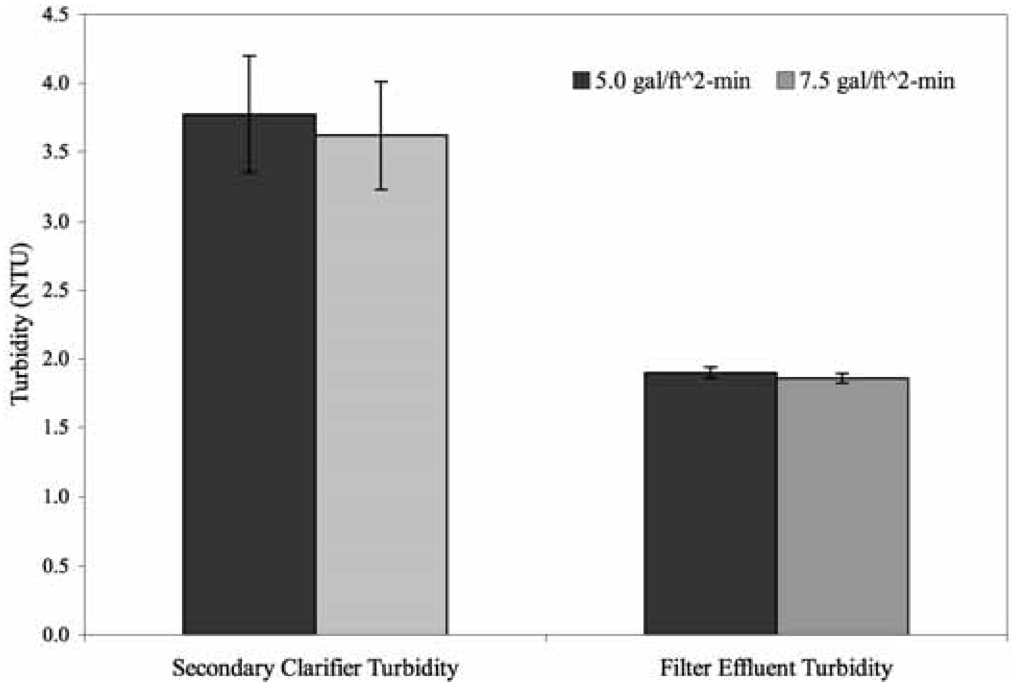


Figure 3.27. Comparison of filter influent and effluent turbidities for filter operation at 5.0 versus 7.5 gal/ft^2-min. while optimizing coagulant dose specific to loading rate (SRO).

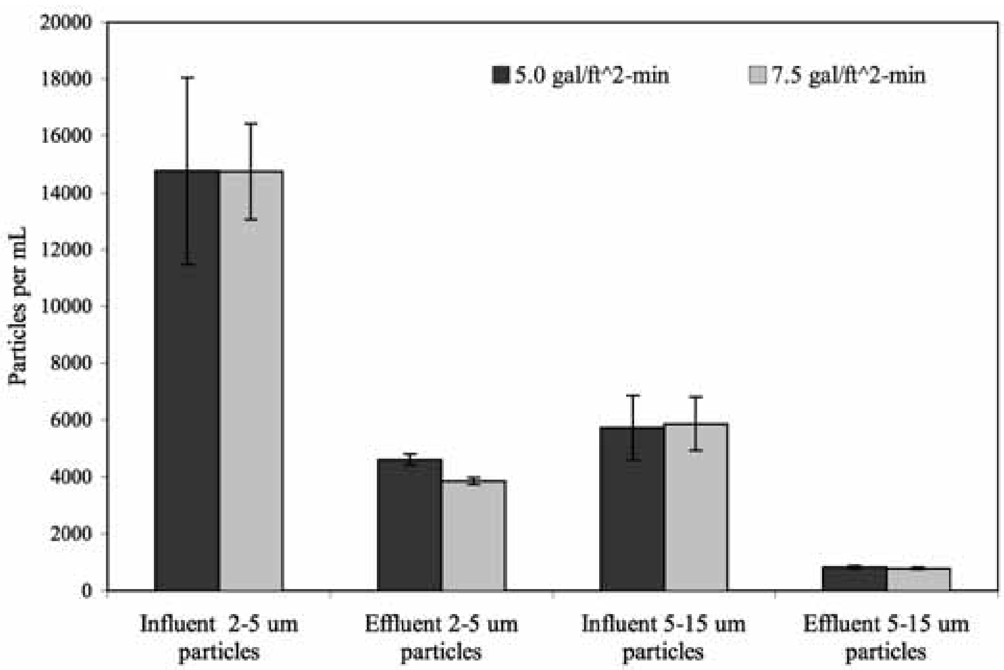


Figure 3.28. Comparison of filter influent and effluent particle counts for filter operation at 5.0 versus 7.5 gal/ft^2-min. while optimizing coagulant dose specific to loading rate (SRO).

3.5 HEAD LOSS AND PARTICLE ACCUMULATION THROUGH THE FILTER BED AS A FUNCTION OF LOADING RATE AND COAGULANT DOSE

The measured clean bed head loss was similar to that predicted by the Carman-Kozeny and the Ergun equations (AWWA, 1999) at all filter loading rates for the three ISP test periods, as shown in Table 3.16. As shown earlier in Figure 3.1, the head loss increased gradually throughout the filter runs until terminal head loss was reached. One impact of loading rate predicted by filtration theory and verified in fundamental filtration experiments is that particles penetrate further into the filter bed as the loading rate increases. To investigate this phenomenon, the pressure measurements at various depths in the filter bed were used to monitor changes in head loss during the filter runs.

To normalize the data for filter loading rate, the head loss (ΔH) for each media section was divided by the clean bed head loss (ΔH_0) of that section. The normalized increase in head loss can then be calculated as $\Delta H/\Delta H_0 - 1$, which is proportional to the mass of deposited particles (Mays and Hunt, 2005). In Figure 3.29, this normalized increase in head loss is plotted as a function of the bed depth (y-axis) to provide a qualitative comparison of particle removal patterns for the five loading rates during a series of ISP runs. The deposition profiles were calculated at a filter throughput of 245 m^3/m^2, which was close to shutdown for most filter runs.

Several important observations about the effect of loading rate can be made from this analysis. For all loading rates tested, the majority of removal occurred in the top few inches of media, consistent with the current understanding of particle removal mechanisms. Based on visual observation, it also appeared that the largest flocs may have been removed by straining at the top surface of the media (Jegatheesan and Vigneswaran, 2005). A time-series analysis of deposition profiles throughout the filter runs revealed that the first measurable particle removal occurred in the top section of the media, that the deeper sections became more important as the runs proceeded, and that this trend was similar for all loading rates (data not shown).

In each media section, $\Delta H/\Delta H_0 - 1$ was inversely proportional to filter loading rate, indicating that higher loading rates decreased the mass removed throughout the filter bed, consistent with the effluent quality data. For higher loading rates, decreased particle deposition at the top of the media resulted in higher particle concentrations in deeper media sections; because particle removal is a function of particle concentration, the deeper sections experienced increased particle capture. This phenomenon may explain why the differences between loading rates decreased deeper in the filter bed. Others have reported that higher loading rates led to a larger share of particle removal occurring deeper in the filter bed (Darby et al., 1991; Kau and Lawler, 1995; Mays and Hunt, 2005).

Only minimal removal of particle mass was observed in the sand, as compared to the anthracite. Nonetheless, the sand may play an important role in the removal of small particles, which are higher in concentration but represent less mass. The deposition profiles overlook this important distinction.

The head loss profile for the SRO runs is presented in Figure 3.30. Similar to the ISP profiles, the 7.5 gal/ft^2-min loading rate exhibited lower head loss in the top section. However, in deeper sections, the 7.5 gal/ft^2-min rate had higher head loss. This key difference is consistent with the fact that the two loading rates had equivalent turbidity and particle removal, so the higher loading rate compensated for lower removal in the upper sections by achieving higher removal in the lower sections. Because the higher coagulant dose used for the 7.5 gal/ft^2-min rate also resulted in greater floc formation, the particle removal at the higher rate was actually higher.

Table 3.16. Average Clean Bed Head Loss (ft) During Different Testing Periods Versus Predicted Values.

Rate (gal/ft^2-min)	Carman-Kozeny	Ergun	ISP1	ISP2	ISP3
2.5	0.7	0.6	0.5	--	--
5.0	1.3	1.2	1.0	1.2	1.1
6.25	1.7	1.5	--	1.5	1.3
7.5	2.0	1.8	1.6	1.8	1.6
8.75	2.3	2.1	--	2.1	1.9
10.0	2.7	2.4	2.2	2.5	2.3
12.5	3.3	3.1	2.8	--	--

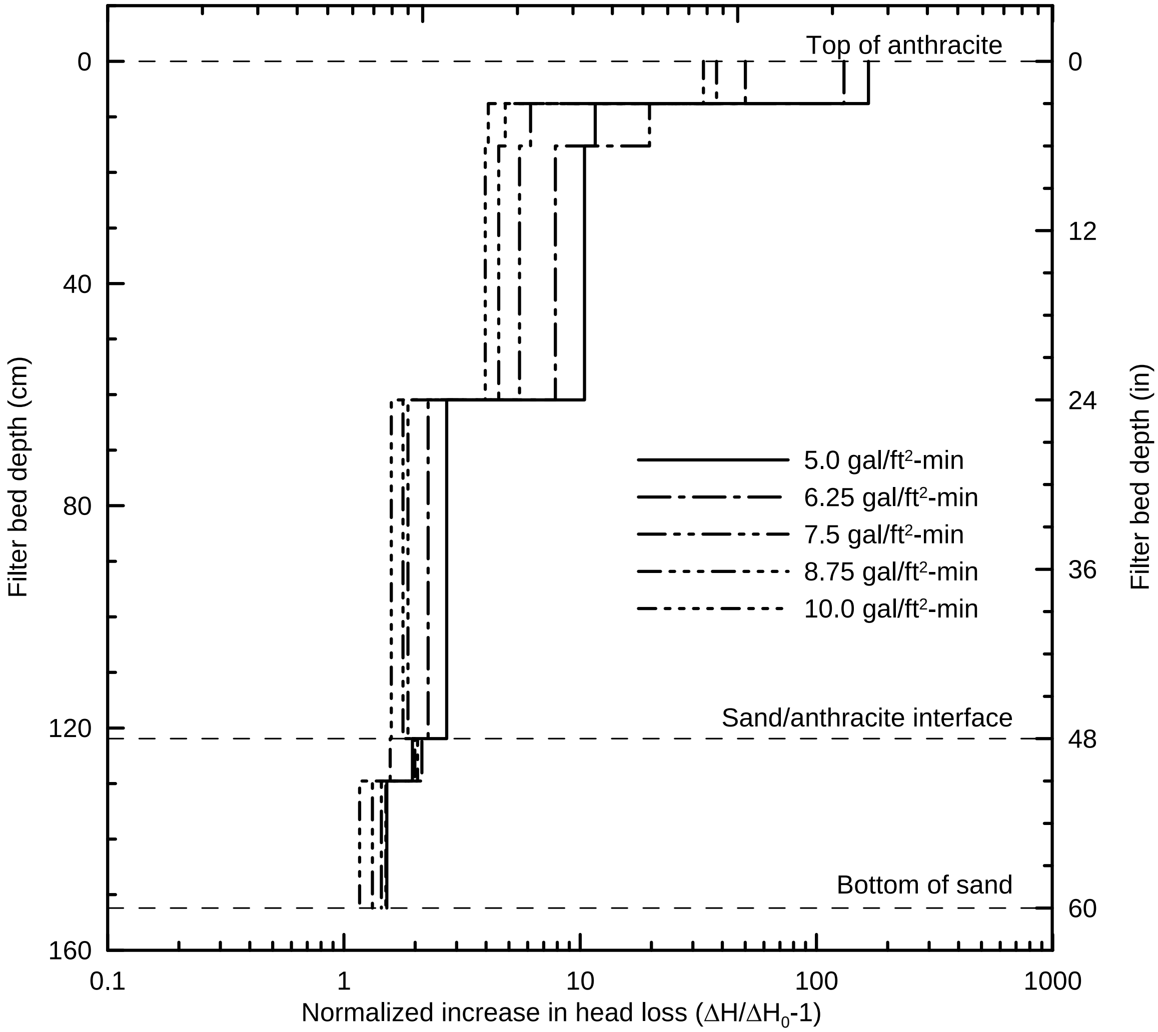


Figure 3.29. Qualitative assessment of accumulation of head loss during the ISP testing, where all loading rates received the same coagulant dose in filter pretreatment.

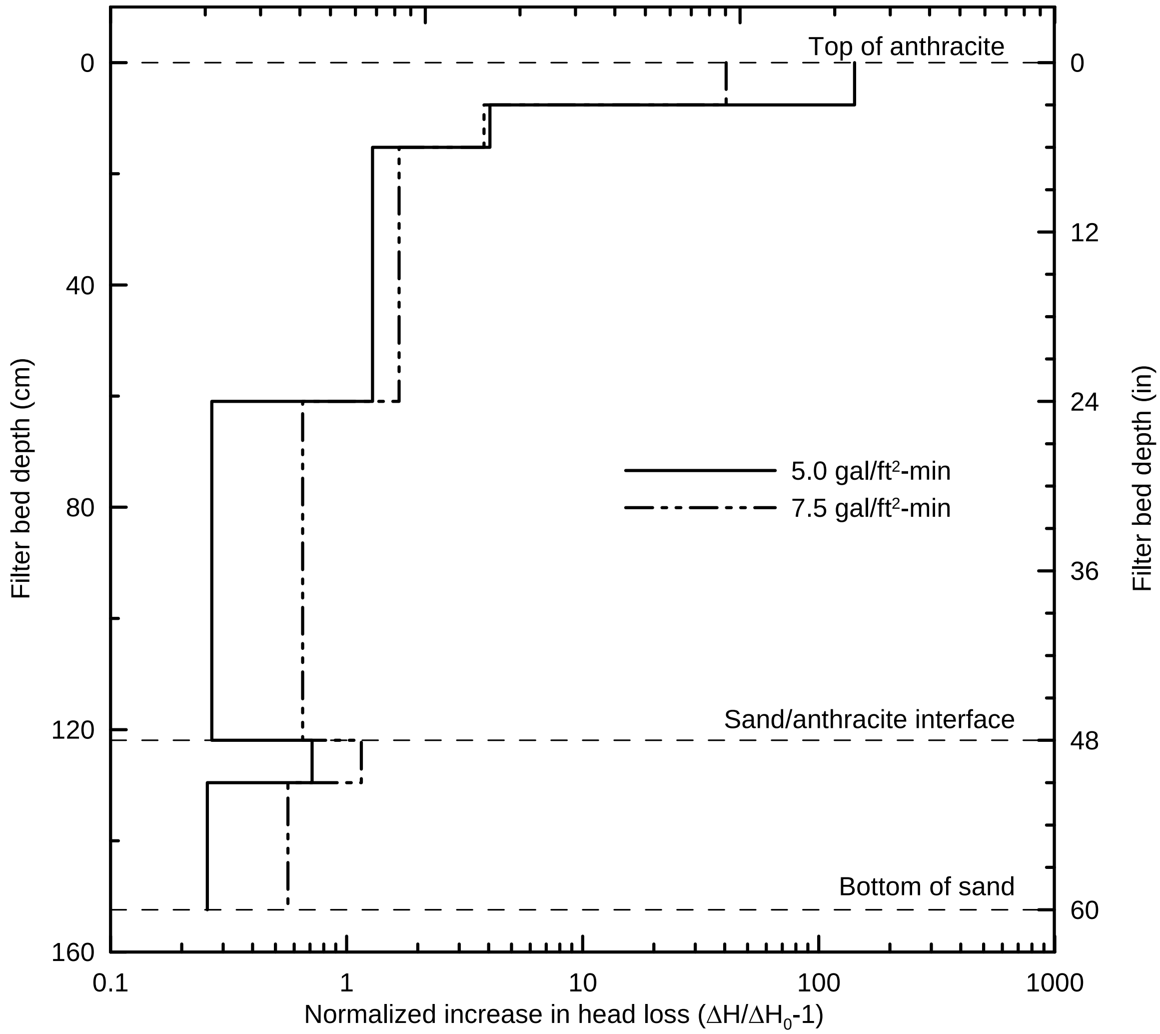


Figure 3.30. Qualitative assessment of accumulation of head loss during the SRO testing, where coagulant dose was optimized specific to the loading rate to produce an equivalent filter effluent.

3.6 DISINFECTION RESULTS

A summary of the batch disinfection results for the three ISP test periods and all loading rates is provided in Tables 3.17 and 3.18. No statistically significant differences were observed between loading rates during any test period. However, it was surprising that a few samples were found to be positive for total coliform bacteria and *E. coli*. To further investigate this phenomenon, a subset of samples was filtered through a 5-µm glass-fiber membrane filter prior to disinfection. As seen in Figures 3.31 and 3.32, in the unmodified samples, although most bacteria were rapidly inactivated, a small number of bacteria was found even after exposure times of up to 120 min and a chlorine residual of 10 mg/L. In the membrane-filtered samples, however, no organisms were detected after the initial inactivation period. Thus, it was concluded that a small number of organisms was embedded in particles that could not be penetrated by disinfectant (chloramine). It is interesting to note that the full-scale plant does not detect coliform bacteria in its disinfected effluent. One key difference between the pilot and full-scale plants is that the full-scale plant practices chlorine addition prior to coagulation to control algae growth in the filters. This pre-chlorination may have the added benefit of inactivating microorganisms before they become incorporated into floc particles that form during coagulation.

Because UV disinfection is becoming more common for disinfecting wastewater effluents, the UV transmittance was investigated in pilot filter effluents. As shown in Table 3.19, no differences were observed with an increase in loading rate, despite the higher turbidities at the higher loading rates.

Table 3.17. Median Total Coliform Count After Batch Chlorine Disinfection. (positive coliform values were due to the particle associated bacteria)

Filtration Rate (gal/ft^2-min)	ISP1 w/ 450 CT (MPN/100 mL)	ISP2 w/1200 CT (MPN/100 mL)	ISP3 w/1200 CT (MPN/100 mL)
2.5	13.7	--	--
5.0	84.7	<2.2	3.0
6.25	--	<2.2	<2.2
7.5	20.4	<2.2	5.2
8.75	--	<2.2	<2.2
10.0	16.0	<2.2	6.3
12.5	11.1	--	--

Table 3.18. Median *E. coli* Count After Batch Chlorine Disinfection

Filtration Rate (gal/ft^2-min)	ISP1 w/ 450 CT (MPN/100 mL)	ISP2 w/1200 CT (MPN/100 mL)	ISP3 w/1200 CT (MPN/100 mL)
2.5	<2.2	--	--
5.0	<2.2	<2.2	<2.2
6.25	--	<2.2	<2.2
7.5	<2.2	<2.2	<2.2
8.75	--	<2.2	<2.2
10.0	<2.2	<2.2	<2.2
12.5	<2.2	--	--

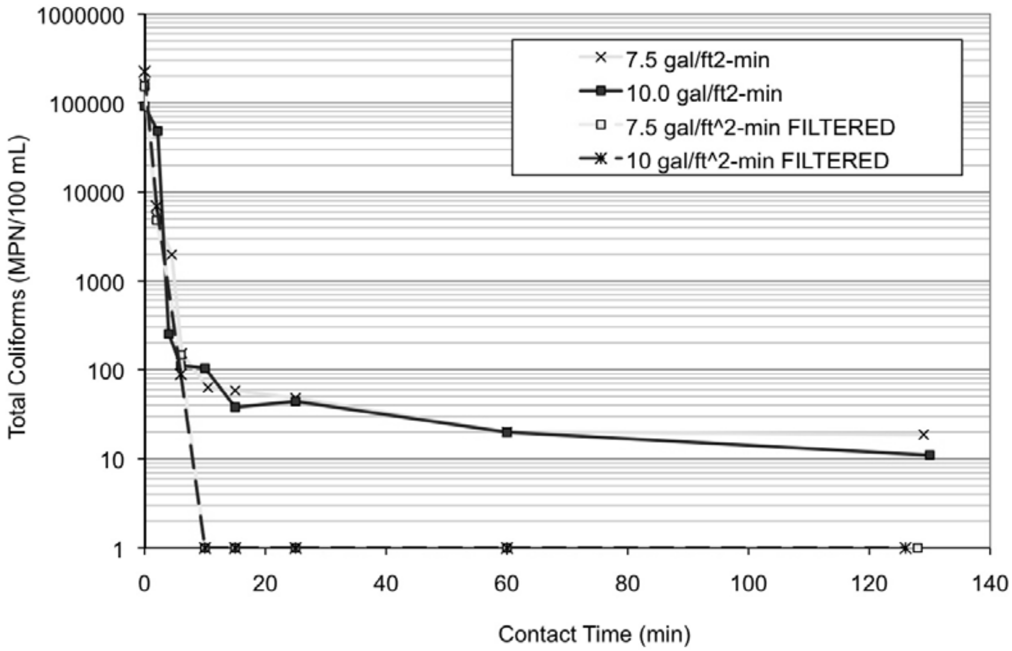


Figure 3.31. Inactivation of total coliform bacteria in pilot-filter effluent at 7.5 and 10 gal/ft^2-min on June 14, 2005. Samples marked as "FILTERED" were passed through a 5 μm glass-fiber membrane filter prior to disinfection. Final chloramine residual of 10 mg/L.

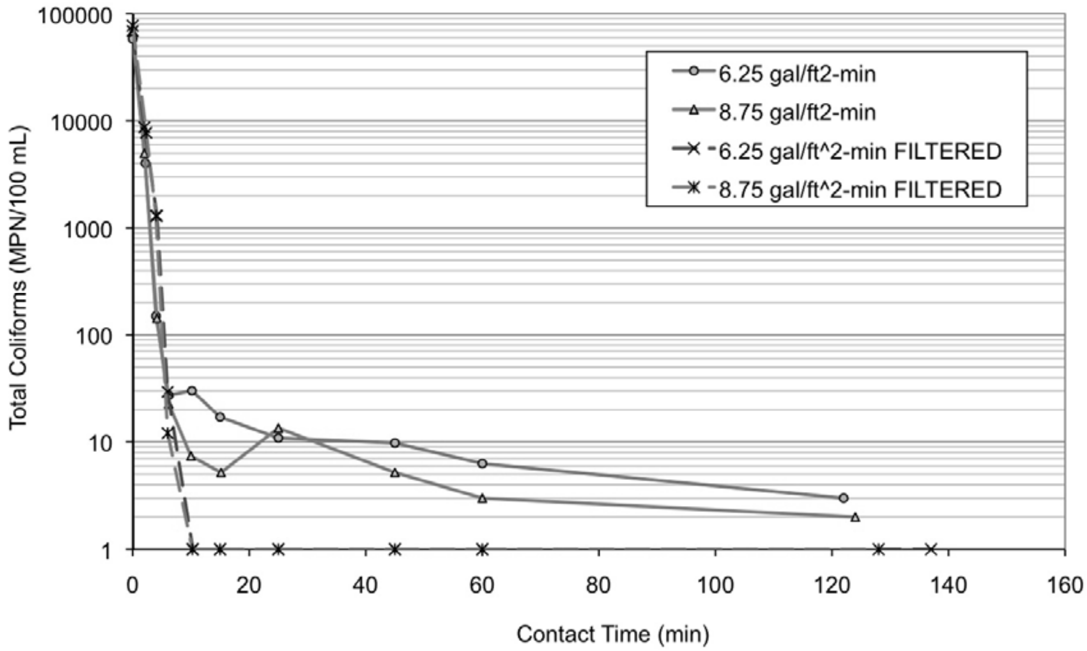


Figure 3.32. Inactivation of total coliform bacteria in pilot-filter effluent at 6.25 and 8.75 gal/ft^2-min on June 28, 2005. Samples marked as "FILTERED" were passed through a 5 μm glass-fiber membrane filter prior to disinfection. Final chloramine residual of 10 mg/L.

Table 3.19. Average UV_{254} Absorbance and Transmittance of Samples Collected (n = 7 per rate) for Disinfection During Third Test Period (ISP3)

Loading Rate (gal/ft^2-min)	UV_{254} Absorbance (mean + 95%CI)	Transmittance (mean + 95%CI)	Turbidity at sampling (NTU) (mean + 95%CI)
5.0	0.177 ± 0.030	66.5% ± 4.6%	1.0 ± 0.5
6.25	0.176 ± 0.015	66.7% ± 2.3%	1.3 ± 0.4
7.5	0.174 ± 0.018	67.0% ± 2.8%	1.4 ± 0.5
8.75	0.179 ± 0.019	66.2% ± 2.9%	1.6 ± 0.6
10.0	0.180 ± 0.016	66.2% ± 2.4%	1.7 ± 0.4

3.7 ASSESSMENT OF EQUIVALENCY FOR PILOT-FILTER RUNS

One of the primary goals of Phase I of the FLEWR project was to provide evidence of equivalent performance at loading rates higher than 5.0 gal/ft^2-min at the pilot plant so that permission could be sought to test higher loading rates at full-scale treatment facilities. A set of criteria were developed for demonstrating equivalent performance at a higher filter loading rate, compared to performance at 5.0 gal/ft^2-min, which was defined as standard practice. These equivalency criteria were developed with input from the California Department of Public Health as well as the Technical Advisory Committee for the FLEWR project. The criteria were:

1. No increase in average effluent turbidity.
2. No increase in effluent particle counts in the size ranges of 2–5 and 5–15 µm.
3. No decrease in log removal of MS2 phage.
4. No decrease in log inactivation of total coliform bacteria through subsequent disinfection.

The equivalency criteria were applied to the SRO filter runs. The evidence for meeting criteria one through three is summarized in Table 3.20 (shown in bold), along with supporting information. It was important to demonstrate that an acceptably small difference between the 5.0 and 7.5 gal/ft^2-min rates could be detected statistically; this value is reported in the fifth column. The evidence for meeting criteria four was presented in the previous section.

Table 3.20. Meeting the Phase I Equivalency Criteria During SRO Test Period

Parameter	5.0 gal/ft^2-min	7.5 gal/ft^2-min	% Change to 7.5	Detection ability	7.5 gal/ft^2-min Performance[8]
Average Volume Treated (gal/ft^2)	7,200	10,200	**42%**	--	--
Secondary Turb (NTU)	3.77	3.62	-4%	--	--
Chemical Dose (mg/L)	3.5	5.6	**62%**	--	More
Inf. Turbidity (NTU)	5.03	5.36	7%	--	--
Eff. Turbidity (NTU)	**1.90**	**1.86**	**-2%**	**4%**	**No Difference**
Inf. Particle Count (2–5 um) (part/mL)	14,800	14,700	-1%	--	--
Eff. Particle Count (2–5 um) (part/mL)	**4,600**	**3,900**	**-15%**	**7%**	**Better**
Log Removal Particles (2–5 um)	0.50	0.58	**16%**	9%	Better
Inf. Particle Count (5–15 um) (part/mL)	5,730	5,860	2%	--	--
Eff. Particle Count (5-15 um) (part/mL)	**830**	**790**	**-5%**	**11%**	**No Difference**
Log Removal Particles (5–15 um)	0.84	0.87	**4%**	7%	No Difference
Log Removal Total Coliforms	0.28	0.29	**3%**	42%	No Difference
Log Removal *E. coli*	0.06	0.25	**298%**	133%	Better
Log Removal MS2 Coliphage	**0.29**	**1.48**	**407%**	**124%**	**Better**

Note. Values shown in bold indicated evidence used for meeting the equivalency criteria defined in Section 3.7.

[8]As statistically compared to performance of the 5.0 gal/ft^2-min rate

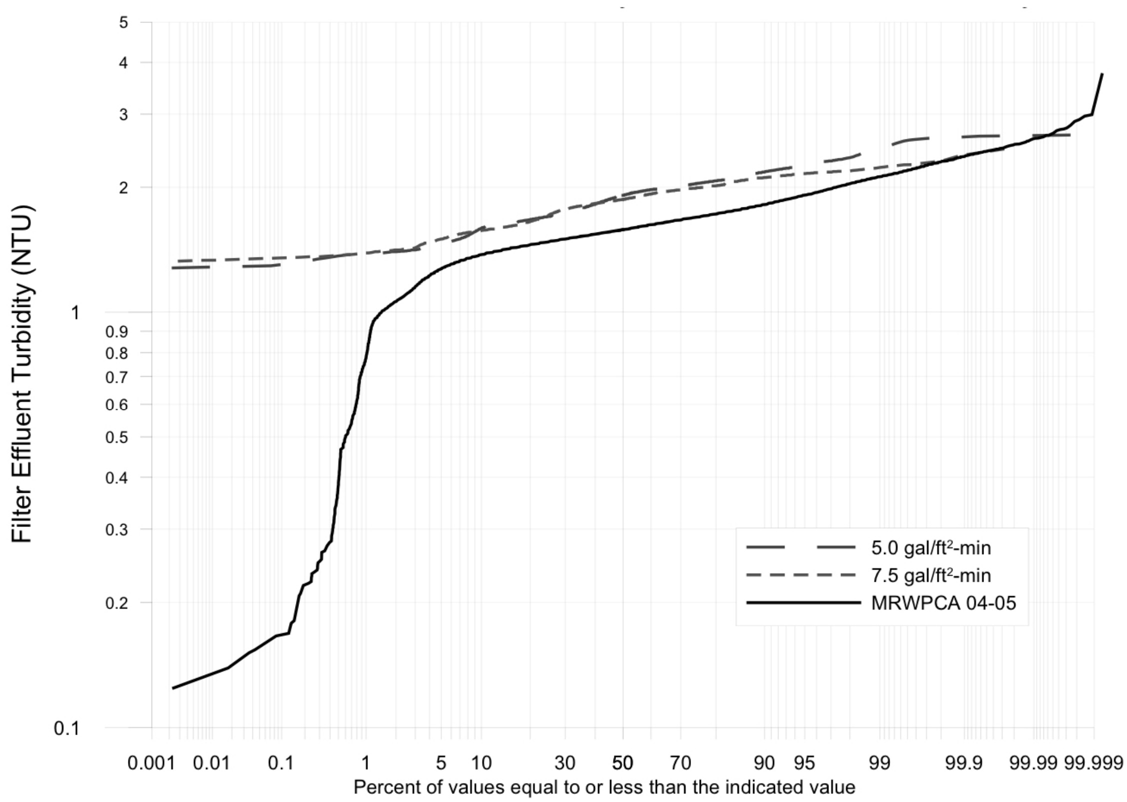


Figure 3.33. Probability plot of filter effluent turbidities during the single loading rate coagulation optimization study period (SRO) compared with Monterey full-scale filter effluent from 2004–2005.

One concern with higher loading rates and with using the pilot scale data to predict performance at the full-scale plant was whether the variability in treatment efficiency at the pilot scale adequately represented that experienced at the full scale. To address this question, in Figure 3.33 a probability plot is presented showing the distributions of filter effluent turbidity data collected for the MRWPCA full scales filters in 2004–2005 (solid line) and the FLEWR pilot filters during the Single Rate Optimization (SRO) data collection period (two dashed lines).

The full scale data were obtained through the random sampling of approximately 10% of the days that the filters were operating in 2004 and 2005 (42 out of 442 days). For each of the selected days, all data points (collected every minute) for each of the six filters were compiled. Data collected when the filter was operating at a flow rate of <0.05 MGD were removed to eliminate data recorded during filter backwash and when not in operation. The distribution of the compiled data was analyzed in a statistical program (JMP IN v.4.0), and probability scores were assigned for each turbidity. The results for this analysis are plotted in Figure 3.33.

The pilot-scale data are the filter effluent turbidity values obtained during the SRO runs in September/October 2005. This period was selected because pilot filter operation was the most similar to the full scale, because the chemical dose was optimized in real time. The data from the first 30 min of each run were excluded because of the low effluent values that were a result of flushing the backwash water out of the filter. The data were analyzed in the same method described for the full-scale data.

As seen in Figure 3.33, the slope of the probability lines is similar for both loading rates at the pilot-scale as well as at the full-scale treatment plant. The high turbidity values at the full-scale plant are worthy of comment. All full-scale data were below 3.0 NTU, except for two one-minute data points (3.7 and 5.0 NTU). In addition, 2.7% of the data was above 2.0 NTU. There were a total of 275,991 data points used in the analysis for the full scale, with 7,347 points above 2.0 NTU. If we assume that each point represents one minute of filter operation, this translates to 122 h (5 d) of filter operation above 2.0 NTU out of the 4,600 h (192 d) surveyed.

As previously reported, there was no statistically significant difference between the average effluent turbidity at the pilot plant between the 5.0 and 7.5 gal/ft^2-min loading rates during the SRO period. The 5.0 gal/ft^2-min rate had a slightly larger range of values (1.28 to 2.68 NTU) than the 7.5 rate (1.33 to 2.51 NTU), and the median values were similar (1.92 and 1.87 NTU respectively).

The low turbidity values (< 1 NTU) for the full-scale system were a result of low flow filtration and recirculation of reclaimed water through the filters on days toward the end of the growing season, when demand for reclaimed water was low.

CHAPTER 4

OVERVIEW OF PHASE II ACTIVITIES

The Phase I results from the pilot-scale experiments were used as an evidence base for pursuing experiments at full-scale treatment plants to compare the performance of 5.0 and 7.5 gal/ft^2·min loading rates under actual operating conditions.

4.1 PHASE II PARTICIPANTS

Five municipal agencies were invited to participate in Phase II activities:

- Monterey Regional Water Pollution Control Agency (MRWPCA)
- Santa Rosa Subregional Water Reuse System
- City of San Jose Water Pollution Control Plant
- Sanitation Districts of Los Angeles County (LACSD)
- Delta Diablo Sanitation District (DDSD)

One of the benefits of conducting filter loading rate experiments at the full scale is that every facility has a unique combination of unit treatment processes, design features, and operations. Thus, Phase II provided an opportunity to explore the impact of loading rate under a range of different conditions. Some key differences between the Phase II treatment plants are highlighted in Table 4.1.

To date, full-scale experiments have been completed at two facilities: MRWPCA and the City of San Jose. Full-scale experiments are underway at DDSD. Regulatory approval for full-scale experiments is in process for Santa Rosa and LACSD.

4.2 EQUIVALENCY CRITERIA

The full-scale portion of the study (Phase II) was designed to compare filter performance at 7.5 and 5 gal/ft^2-min. As previously agreed on with the California Department of Public Health (CDPH), the results from each plant participating in the study are evaluated using a set of criteria to determine if the filters operated at both rates receive an equal degree of treatment. The equivalency criteria for Phase II of the project are:

1. No significant* increase in mean turbidity of filter effluent.
2. No significant* increase in mean concentration of 2–5 and 5–15 μm particles in filter effluent.
3. No significant decrease in the ability to disinfect filter effluent.

*Where significant increase = $\dfrac{0.2\ \text{NTU}}{\text{NTU produced at 5.0 gal/ft}^2\text{-min}}$ (reported as percent)

Table 4.1. Full-Scale Treatment Plants Participating in Phase II Activitie.

Parameter	**MRWPCA**		**Santa Rosa**	**San Jose**		**LACSD**		**DDSD**
Secondary	Tricking filter and bioflocculation		Activated Sludge	Activated Sludge		Activated Sludge		Trickling Filter and Activated Sludge
Filter Pretreatment	Coag/Floc		Coag	Coag		Coag		Coag/Floc/Sed
Filter Inf. Turbidity (NTU)	4		1.3	1.6				1.3
Filter Eff, Turb (NTU)	1.9		0.7	0.6				<1
Media Type	Sand	Anth	Anth	Sand	Anth	Anth	Sand	Sand
Media depth (ft)	1	4	4	1	1.8	2	1	8

4.3 EXPERIMENTAL PROTOCOLS

The experimental protocols for Phase II operations were developed individually for each treatment plant. As with the pilot facilities, online instruments were used to measure turbidity and particle removal across the filters, and grab samples were collected for analysis of total coliform bacteria and *E. coli*. The ability to disinfect filter effluents was assessed with batch tests in the laboratory that mimicked full-scale disinfection practices. A key difference between pilot- and full-scale experiments is that it was not possible to seed MS2 coliphage before the full-scale filters because of the large volumes of virus culture that would have been needed. However, laboratory disinfection tests were performed with MS2.

An important emphasis at the full-scale facilities was to assess the feasibility of the higher loading rates in terms of filter run length, filter production capacity, coagulant doses, clean bed head loss, and any changes to backwash requirements.

At San Jose, simultaneous testing was performed, in which two test filters received the same influent water but were operated at different loading rates. At Monterey it was not possible to operate two filters simultaneously because of insufficient nighttime flow. Thus, consecutive testing of loading rates was performed, in which only one test filter was used and the test loading rate was alternated from run to run.

A detailed description of the operating protocols for each full-scale facility is provided in Appendix C.

CHAPTER 5

IMPACT OF LOADING RATE ON FULL-SCALE TREATMENT

5.1 IMPACT OF LOADING RATE AT FULL-SCALE MONTEREY

Full-scale filter tests at the MRWPCA treatment plant were conducted over 18 months. Sixty-two filter runs were completed (31 at each loading rate) spanning the three seasons that MRWPCA produces reclaimed water, and the necessary statistically rigors of the FLEWR study were achieved.[9]

An initial series of 10 filter runs was completed in Summer 2007, during which operators optimized the coagulant dose such that the filter effluent would remain around 1.8 NTU at both loading rates. However, under these conditions, the effluent particle counts (both 2–5 and 5–15 µm ranges) were higher when the filters were operated at 7.5 versus 5 gal/ft^2-min (see Appendix D). The coagulation strategy was then changed for FLEWR testing, such that operators optimized the coagulant dose to produce a lower filter effluent turbidity for the 7.5 versus the 5 gal/ft^2-min rate. Because these preliminary runs were performed under different operating conditions from the rest of the MRWPCA testing, they were not used in the analysis to determine equivalency. However, it is important to note that even if these runs were included in the overall analysis, the equivalency criteria and the statistical rigors of the FLEWR study would still be met.

Seven runs could not be included in the analysis for various reasons. These reasons included instrumentation malfunction, operator error, and plant upsets (see Appendix D). Because only one loading rate could be tested at a time, the impact of these events was specific to one loading rate, and these data were not used in the overall analysis to avoid biasing the results.

The following is a summary of the results.

5.1.1 Removal of Turbidity and Particles

Turbidity and particle counts were monitored and recorded every minute for the secondary effluent, coagulated/flocculated filter influent[10], and the test filter effluent. The mean value for each parameter was determined for each run (Appendix D) and the overall mean for each rate was then calculated (Tables 5.1 and 5.2). Statistical analyses were performed on the mean values for each loading rate. Probability plots of turbidity and particle counts (2–5 and 5–15 µm ranges) provide additional detailed comparisons (Appendix D).

A comparison of the overall performance of filters operated at the two different loading rates is provided in Figure 5.1. The average filter effluent turbidity and the average particle counts in the 2–5 µm range were lower at the 7.5 gal/ft^2-min rate than at the 5.0 gal/ft^2-min rate. The average particle counts in the 5–15 µm range were equal at the two loading rates.

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[9] The least significant difference was less than or equal to the equivalency criteria definition of significant (i.e. the least significant difference for all equivalency parameters was $\leq$ 11.3%; see Appendix D).
[10] Coagulated/flocculated monitoring equipment was installed after the preliminary test period and became operational prior to Run 2 (9/11/2007).

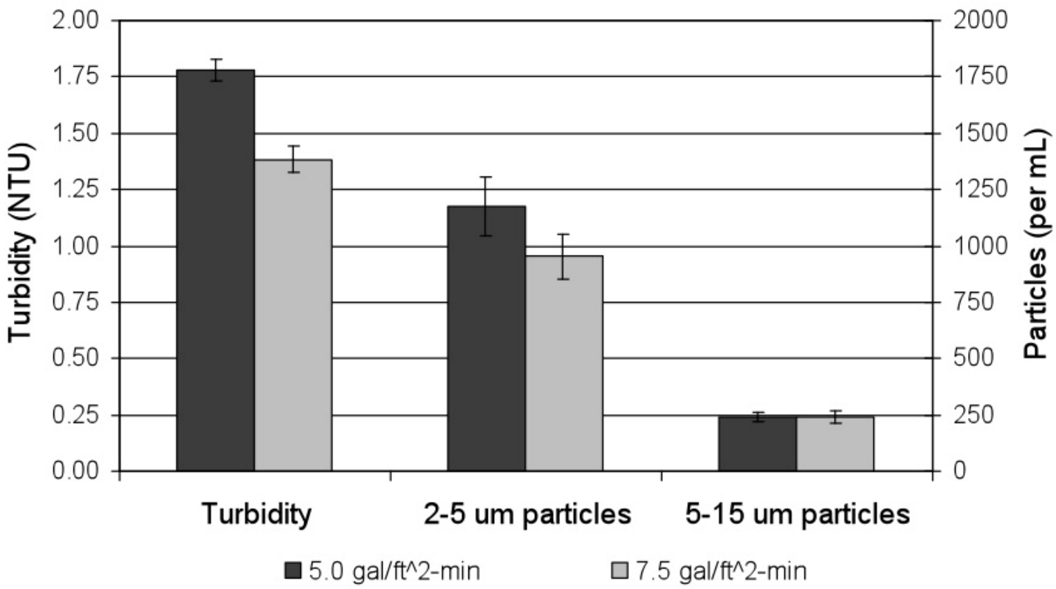

Figure 5.1. Comparison of particle concentration metrics in MRWPCA filter effluent. Error bars indicate the 95% confidence interval for the plotted mean value.

Even though it was only possible to test one loading rate at a time, the secondary effluent water quality during the FLEWR testing was statistically equivalent for the two loading rates. The mean secondary effluent turbidity was slightly higher during 5 gal/ft^2-min testing, but the secondary effluent particle counts were lower for the 5 gal/ft^2-min rate (see Tables 5.1 and 5.2).

Table 5.1. Run Characteristics and Turbidity Data From Monterey Full-Scale Filter Loading Rate Testing

Loading Rate (gal/ft^2-min)	Number of Runs (n)	Run Time (h)	Coagulant Dose (mg/L)	Turbidity (NTU)		
				Secondary	Filter Influent	Filter Effluent
5.0	31	22.6 ± 0.73	5.1 ± 0.6	4.05 ± 0.22	7.00 ± 0.39	1.78 ± 0.05
7.5	31	12.0 ± 0.61	7.7 ± 0.8	4.00 ± 0.30	7.41 ± 0.43	1.38 ± 0.06
%Δ from 5.0–7.5	--	-47%	51%	-1%	6%	-22%

Note. Values given are mean + 95% confidence on mean. Bolded % indicates parameter is statistically different between loading rates 5.0 and 7.5 gal/ft^2-min.

A 51% increase in coagulant usage was needed to achieve equivalent performance when operating at 7.5 gal/ft^2-min. The flocculation process reduced the overall number of particles, as many smaller diameter particles agglomerated to form larger particles. The higher coagulant doses at 7.5 gal/ft^2-min caused a significant increase in 2–5 µm particles and turbidity in the filter influent, as compared to 5 gal/ft^2-min doses, but this shift also led to an increase in the overall filter performance in terms of particle removal through the filter at the higher rate (see Figure 5.1).

The improved filter performance at the 7.5 gal/ft^2-min rate (versus the 5.0 gal/ft^2-min rate) is reflected in the difference in filter effluent turbidities. The filter effluent turbidity at 7.5 gal/ft^2-min decreased by 22% (or 0.4 NTU; see Table 5.1). This higher performance also led to a 19% (or 220 particles/mL) decrease in the number of 2–5 µm particles in the filter effluent (see Table 5.2). The mean number of particles in the 5–15 µm size range was the same for both loading rates (see Table 5.2). All three of these data sets are well below the 11.3% increase defined as significant by the previously defined equivalency criteria.

Table 5.2. Particle Count Data from Monterey Full-Scale Filter Loading Rate Testing

Loading Rate (gal/ft^2-min)	2–5 mm Particles (1000 per mL)			Particles 5–15 mm (100 per mL)		
	Secondary	Filter Influent	Filter Effluent	Secondary	Filter Influent	Filter Effluent
5.0	13.7 ± 1.2	8.9 ± 1.1	1.17 ± 0.13	49.9 ± 5.4	49.6 ± 4.5	2.39 ± 0.02
7.5	14.4 ± 1.2	11.0 ± 1.6	0.95 ± 0.10	52.3 ± 5.6	47.0 ± 4.0	2.39 ± 0.03
%D from 5.0-7.5	4.6%	**24%**	**-18.9%**	4.7%	-5.3%	0%

Note. Values given are mean + 95% confidence on mean. Bolded % indicates parameter is statistically different between loading rates 5.0 and 7.5 gal/ft^2-min.

5.1.2 Disinfection of Filter Effluent

The ability to disinfect the filter effluent, as required by the third equivalency criterion, was tested by performing 73 batch coliform disinfection experiments at the MRWPCA laboratory (protocol in Appendix C) and the virus disinfection experiment at UC Berkeley as specified by the Virus

Disinfectability protocol.[11] No decrease in the ability to disinfect the water was found. The following is a summary of the disinfection findings (detailed results reported in Appendix D).

The bench-scale disinfection testing was designed to mimic the full-scale disinfection. Because MRWPCA practices pre-chlorination prior to tertiary filtration, coliform concentrations going into the filters were typically low. Filter effluent samples were collected and immediately taken to the laboratory for the disinfection experiment. Sodium hypochlorite was added to the samples such that the residual at 120 min was approximately 10 mg-Cl/L (a 1200 mg/L-min C*T target). The actual mean CT values were slightly higher than 1200 mg/l-min because of variation in the chlorine residual (see Table 5.3).

In terms of total coliform bacteria disinfection, all samples except those specified in the Table 5.3 footnotes had a most probable number (MPN) less than the 2.2 per 100 mL of sample. The CDPH Water Recycling Criteria specify the 7-day *median* concentration be less than 2.2 MPN/100 mL. Therefore, both rates had adequate disinfection in terms of total coliform bacteria. In addition, 22% and 11% of the disinfection samples tested at 5 and 7.5 gal/ft^2-min, respectively, had coliform concentrations of 1–2 MPN/100 mL. Overall, there was no reduction in the ability to disinfect total coliform bacteria at the higher loading rate. In terms of *E. coli*, only one sample at each filter loading rate had any positive wells (both were 1.0 MPN/100mL;[12] see Table 5.3), thus indicating that there was no decrease in the disinfection of *E. coli* at the 7.5 gal/ft^2-min filter loading rate.

Table 5.3. Results from Bench-Scale Disinfection Tests

Loading Rate (gal/ft^2-min)	Average C*T (mg/L-min)	Total Coliform Bacteria		*E. coli*	
		Positive Samples[13]	$\geq$2.2 MPN per 100 mL[14]	Positive Samples[13]	$\geq$2.2 MPN per 100 mL
5.0	1281	10	2	1	0
7.5	1338	5	1	1	0

Note. 37 tests completed per loading rate.

The virus disinfection protocol used filter effluent from MRWPCA treated at both loading rates. Several chlorine doses were applied to filter effluent samples, and inactivation of seeded MS2 coliphage was measured after 90 min (see Table 5.4). The dose-inactivation rates (slopes of curves in Figure 5.2) for water treated at both loading rates were similar and not statistically different ($\alpha = 0.05$). Because MRWPCA does not have a nitrification step as part of the treatment process, the chlorine added to the samples was rapidly converted to less reactive chloramines. A fourth chlorine dose was selected to achieve breakpoint chlorination and disinfect with free chlorine, where inactivation of coliphage (greater than 7 logs) was observed. Because of the high ammonia concentration, the chlorine dose required to achieve breakpoint chlorination was extremely high (~300 mg Cl/L) so full-scale implementation would not be practical. Regardless,

[11] Submitted by Professor Kara Nelson to CDPH on November 6, 2007
[12] The IDEXX method has been shown to have a significant incidence of false-positives for *E. coli* (Yakub et al. 2002).
[13] IDEXX Colilert has a detection limit of 1 MPN/100 mL (one positive well).
[14] For loading rate 5 gal/ft^2-min: Run 11 (10/1/2007) at 35 MPN/100mL and Run 12 (10/3/2007) at 6 MPN/100mL; for 7.5 gal/ft^2-min: Run 7 (9/24/2007) at 18 MPN/100mL

there was no decrease in the ability to disinfect MS2 virus seeded into the filter effluent at the higher loading rate.

Table 5.4. Results From Virus Disinfection in MRWPCA Filter Effluent

Loading Rate (gal/ft^2-min)	Target CT (mg/L-min)	Chlorine Dose (mg/L)	90-min Chlorine Residual (mg/L)	CT Value (mg/L-min)	Spiked MS2 Log(PFU/mL)	Final MS2 Log(PFU/mL)	Log MS2 Inactivation
5	450	6	5.0	450	7.2	6.9 ± 0.1	0.3
	1200	14	14.1	1300		6.8 ± 0.2	0.5
	2400	27	25.9	2300		6.2 ± 0.1	1.0
	past breakpoint	300	35.0	3100		<0	>7.2
7.5	450	6	5.5	490	7.2	7.0 ± 0.1	0.2
	1200	14	13.6	1200		6.8 ± 0.3	0.5
	2400	27	24.5	2200		6.1 ± 0.1	1.1
	past breakpoint	300	18.7	1700		<0	>7.2

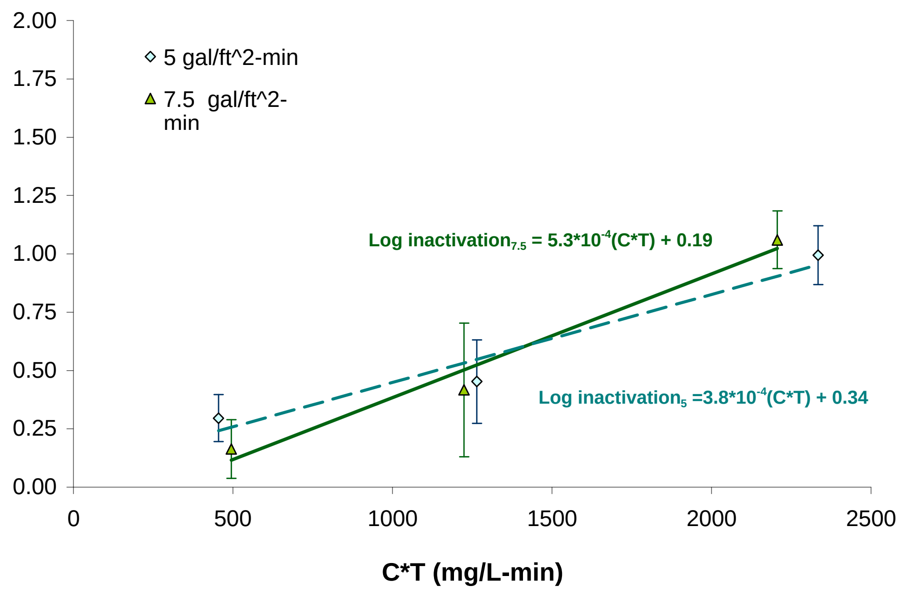

Figure 5.2. Log reduction of MS2 as a function of CT for MRWPCA filter effluent. No decrease in the ability to disinfect MS2 coliphage was detected between the two filter loading rates. Linear regression lines are shown for both rates (solid green/dark line = 7.5 gal/ft^2-min; dashed blue/light line = 5 gal/ft^2-min).

5.1.3 Assessment of Equivalency Criteria

Based on the four Phase II equivalency criteria, the MRWPCA full-scale plant achieved equivalent performance while testing at a 7.5 gal/ft^2-min filter loading rate, as compared to operations at 5 gal/ft^2-min. To accomplish equivalent particle counts in the 5–15 µm size range, it was necessary to use a coagulant dose that was about 50% higher at 7.5 gal/ft^2-min than at 5 gal/ft^2-min. No difference in disinfection ability was detected through the bench-scale disinfection experiments.

5.2 IMPACT OF LOADING RATE AT FULL-SCALE SAN JOSE

Full-scale filter tests at the San Jose treatment plant were conducted in two test periods. A preliminary test period was conducted during March 2007, consisting of 40 filter-runs (20 at each rate) and formal FLEWR runs conducted in August 2007, which consisted of 30 filter-runs (15 at each rate). Several issues related to instrumentation were identified after the preliminary filter runs and were corrected for the August test period. The necessary statistical rigors of the FLEWR study were achieved.[15] The protocols used during the testing period are provided in Appendix D. At no time during the testing was it required to add coagulant to the secondary effluent prior to filtration.

The following is a summary of the findings.

5.2.1 Removal of Turbidity and Particles

Turbidity and particle counts were monitored and recorded every minute for the secondary effluent, coagulated/flocculated filter influent[16], and the test filter effluent. The mean value for each parameter was determined for each run (see Appendix D) and the overall mean for each rate was then calculated (see Tables 5.5 and 5.6). Statistical analyses were performed on the mean values for each loading rate. Probability plots of turbidity and particle counts (2–5 and 5–15 µm ranges) provide additional detailed comparisons (see Appendix D).

A comparison of the overall performance of filters operated at the two different loading rates is provided in Tables 5.5 and 5.6 and Figure 5.3. The average influent quality was equivalent for filters operated at both loading rates. The average filter effluent turbidity and particle counts in both size ranges were slightly higher at the 7.5 gal/ft^2-min rate than at the 5.0 gal/ft^2-min rate. However, these differences were not statistically significant and were well below the increase defined as significant by the equivalency criteria, which was 27.7%. (Of the equivalency criteria parameters, the maximum difference that could be detected with 95% confidence was 14.6%.)

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[15] The least significant difference was less than or equal to the equivalency criteria definition of significant (i.e., the least significant difference for all equivalency parameters was $\leq$ 27.7%; See Appendix X).
[16] Coagulated/flocculated monitoring equipment was installed after the preliminary test period and became operational prior to Run 2 (9/11/2007).

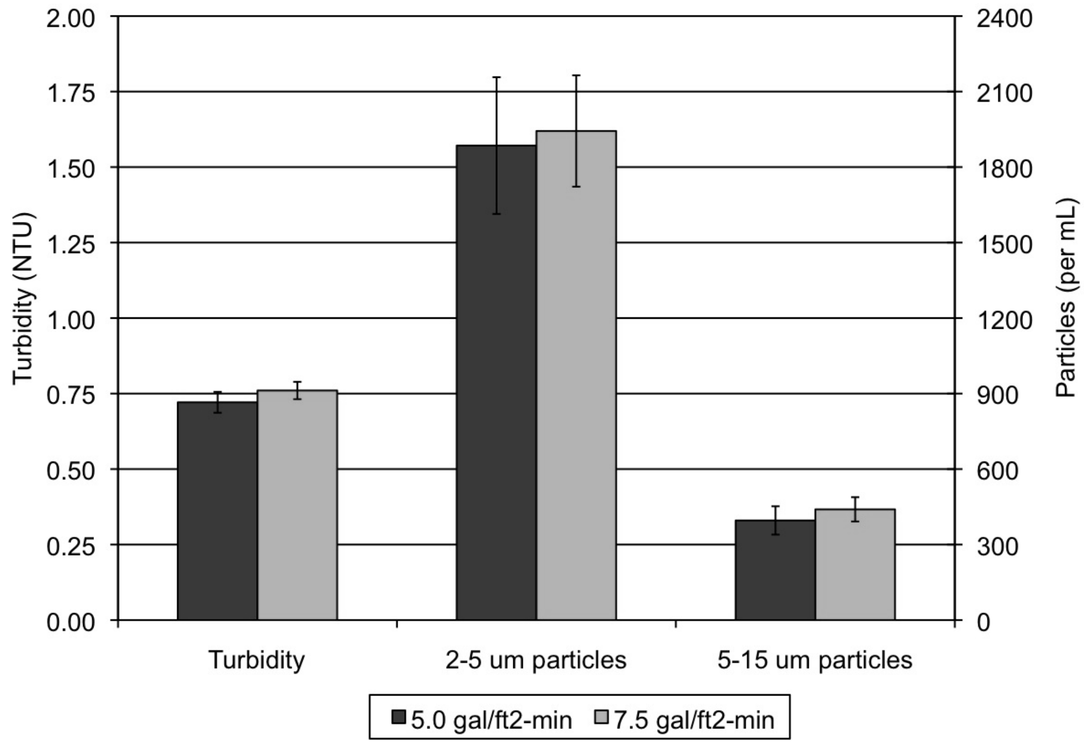

Figure 5.3. Comparison of particle concentration metrics in San Jose filter effluent. Error bars indicate the 95% confidence interval for the plotted mean value.

Table 5.5. Run Characteristics and Turbidity Data from San Jose Full-Scale Filter Loading Rate Testing

Loading Rate (gal/ft^2-min)	Total Number of Runs (n)	Run Time (h)			Turbidity (NTU)					
					Secondary			***Filter Effluent***		
5.0	15	24.8	$\pm$	2.57	1.65	$\pm$	0.21	0.72	$\pm$	0.03
7.5	15	14.4	$\pm$	1.26	1.70	$\pm$	0.25	0.76	$\pm$	0.03
%Δ from 5.0-7.5	--	**-42%**			3%			5%		

Note. Values given are mean + 95% confidence on mean. Run time was the only parameter that was statistically different between the two loading rates (shown in bold).

Table 5.6. Particle Count Data from San Jose Full-Scale Filter Loading Rate Testing

Loading Rate (gal/ft^2-min)	2–5 mm particles (1000 per mL)		5–15 mm particles (100 per mL)	
	Filter Influent	*Filter Effluent*	*Secondary*	*Filter Effluent*
5.0	4.40 ± 0.30	1.89 ± 0.27	16.5 ± 2.0	3.96 ± 0.56
7.5	4.30 ± 0.30	1.94 ± 0.22	16.0 ± 1.4	4.40 ± 0.48
%D from 5.0–7.5	-1.4%	3.1%	-3.3%	11%

Note. Values given are mean + 95% confidence on mean.

5.2.2 Disinfection of Filter Effluent

The ability to disinfect the filter effluent, as required by the third equivalency criterion, was tested by performing 69 batch coliform disinfection experiments at the San Jose laboratory (protocol in Appendix C) and the virus disinfection experiment at UC Berkeley as specified by the Virus Disinfectability protocol.[17] No decrease in the ability to disinfect the water was found. The following is a summary of the disinfection findings (detailed results reported in Appendix D).

The bench-scale disinfection testing was designed to mimic the full-scale disinfection. Thus, batch samples were first disinfected with chloramines for an average CT of 62 to 70 mg/L-min, followed by free chlorine for a CT of 502 to 512 mg/L-min (Tables 5.7 and 5.8). After the second disinfection step (with free chlorine), five and seven samples had a concentration greater than or equal to 2.2 MPN/100 mL for the 5 and 7.5 gal/ft^2-min rates, respectively. The CDPH Water Recycling Criteria specify that the 7-day *median* concentration be less than 2.2 MPN/100 mL. Both rates had adequate disinfection in terms of total coliform bacteria. In terms of *E. coli*, no samples at either filter loading rate had concentrations greater than 2.2 MPN/100 mL. Most important, there was no reduction in the ability to disinfect total coliform bacteria or *E. coli* at the higher loading rate.

Table 5.7. Total Coliform and *E. coli* Detection After Chloramination at San Jose

Loading Rate (gal/ft^2-min)	# Tests	Average CT (mg/L-min)	Total Coliform		*E. coli*	
			Positive Samples	≥2.2 MPN per 100 mL	Positive Samples	≥2.2 MPN per 100 mL
5.0	69	62	61	41	9	1
7.5	69	70	59	43	11	1

[17] Submitted by Professor Kara Nelson to CDPH on November 6, 2007.

Table 5.8. Total Coliform and *E. coli* Detection After Chlorination at San Jose

Loading Rate (gal/ft^2-min)	# Tests	Average CT (mg/L-min)	Total Coliform		*E. coli*	
			Positive Samples	$\geq$2.2 MPN per 100 mL	Positive Samples	$\geq$2.2 MPN per 100 mL
5.0	69	502	22	5	1	0
7.5	69	512	29	7	0	0

The virus disinfection protocol used filter effluents from San Jose treated at both loading rates. Several chlorine doses were applied to filter effluent samples, and inactivation of seeded MS2 coliphage was measured after 90 min (see Table 5.9). MS2 coliphage was not found in any treated samples. Based on the initial concentration of MS2, the inactivation was greater than 6.5 logs in all cases. Thus, there was no decrease in the ability to disinfect MS2 virus seeded into the filter effluent at the higher loading rate.

Table 5.9. Results From Virus Disinfection Study at San Jose

Loading Rate (gal/ft^2-min)	Target CT (mg/L-min)	Chlorine Dose (mg/L)	90-min Chlorine Residual (mg/L)	CT Value (mg/L-min)	Spiked MS2 Log(PFU/mL)	Final MS2 Log(PFU/mL)	Log MS2 Inactivation
5	450	13	2.8	254	6.6	<0	>6.6
5	1200	21	9.1	821	6.6	<0	>6.6
5	2400	35	20.7	1866	6.6	<0	>6.6
7.5	450	13	2.8	250	6.5	<0	>6.5
7.5	1200	21	9.3	840	6.5	<0	>6.5
7.5	2400	35	21.6	1940	6.5	<0	>6.5

5.2.3 Assessment of Equivalency Criteria

Based on the four Phase II equivalency criteria, the San Jose full-scale plant achieved equivalent performance while testing at a 7.5 gal/ft^2-min filter loading rate, as compared to operations at 5 gal/ft^2-min.

5.3 COMPARISON BETWEEN FULL-SCALE TREATMENT PLANTS AND THEORY

The influent turbidity and particle counts were much higher at the tertiary filtration facility in Monterey compared to San Jose (see Figure 5.4). These differences are a result of the upstream treatment processes at each facility (trickling filters/bioflocculation tanks at Monterey, and activated sludge at San Jose). Despite much higher removal efficiencies in Monterey, the Monterey filter effluent turbidity was higher than the San Jose influent turbidity. However, the particle counts in the Monterey effluent were even lower than those in the San Jose effluent (Figure 5.4), and this was true at both loading rates (see Figure 5.5).

A key difference between the two treatment plants is the use of tertiary coagulation: Monterey continually added coagulant prior to filtration, whereas San Jose did not add any coagulant during the testing period.

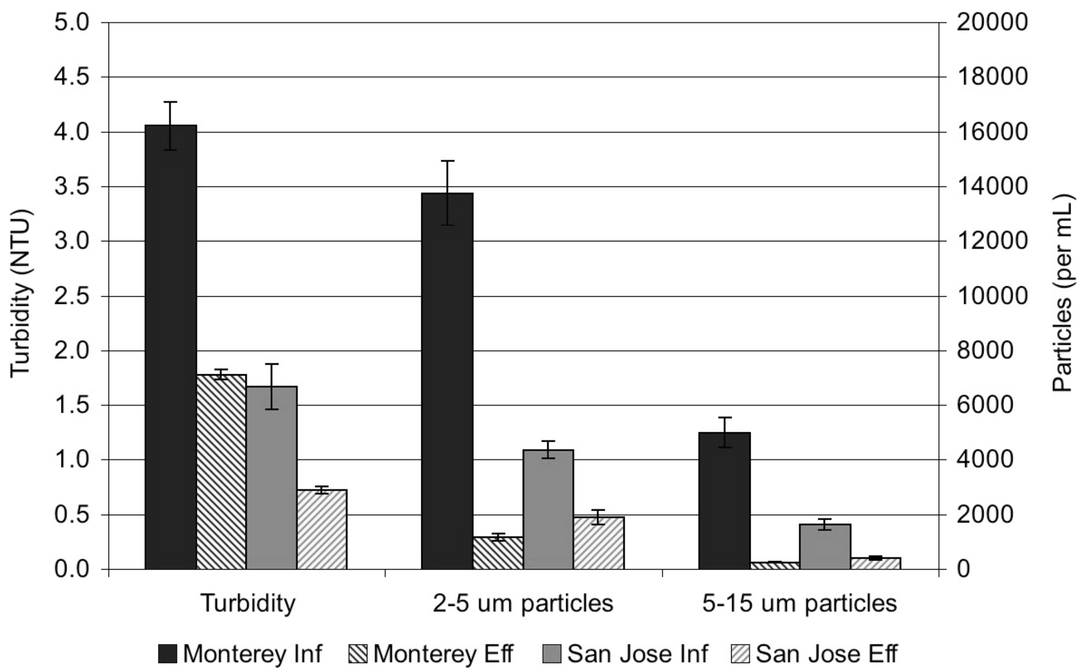

Figure 5.4. Comparison of secondary effluent (inf = solid) and filter effluent (eff = dashed) turbidity and particle counts from full-scale testing at Monterey and San Jose at 5.0 gal/ft^2-min.

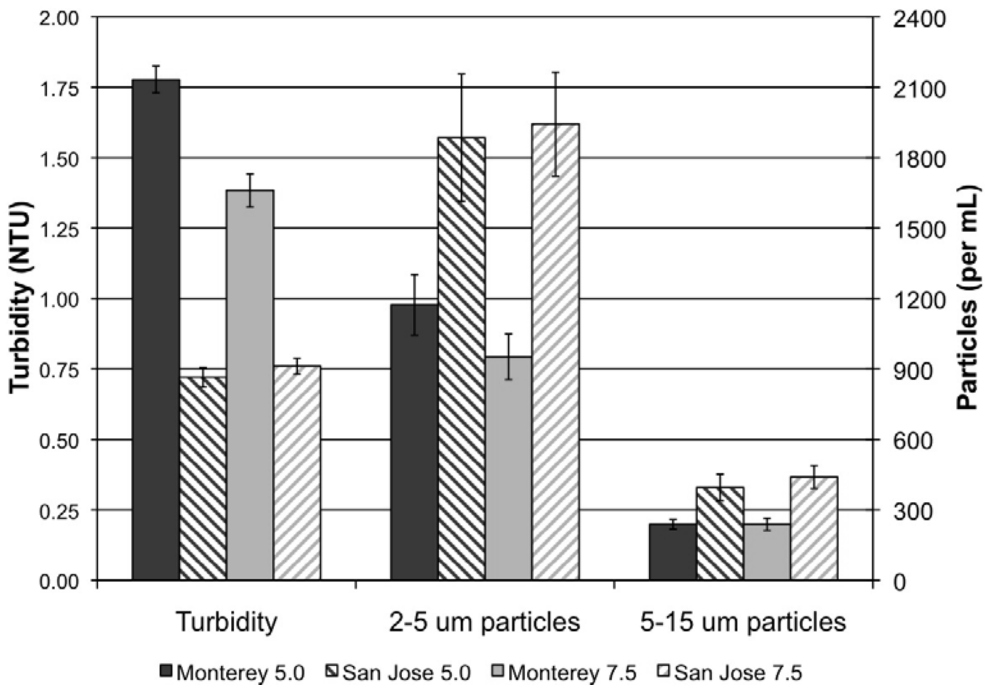

Figure 5.5. Comparison of filter effluent turbidity and particle counts from full-scale testing at Monterey and San Jose at loading rates 5.0 (solid) and 7.5 (dashed) gal/ft^2-min.

The clean bed particle removal efficiencies at the full-scale Monterey filters are compared with those predicted by theory in Figures 5.6 and 5.7 for the two loading rates. As with the pilot-plant data, the value of alpha was set to one. The impact of particle size on removal efficiency was similar at the full-scale and the pilot plant. The particle removal efficiencies at the full-scale San Jose filters are compared with theory in Figures 5.8 and 5.9. It is interesting to note that the removal efficiencies at San Jose agree extremely well with the model predictions. It is important however, that for the San Jose data, the values for alpha were determined based on the best fit. These values were 0.69 and 0.64 for 5.0 and 7.5 gal/ft^2-min, respectively.

The excellent agreement between measurements and theory in San Jose contrast with those in Monterey and suggests that the filtration model does not adequately capture the impact of coagulation. Even with the alpha value set to one for Monterey, significantly better than predicted clean bed removal was observed for the smallest measured particles (2–3 μm), and the difference between predicted and observed values decreased with increasing particle size. Several factors may explain the better than predicted particle removal, including (a) particles remaining on the filter media after backwashing enhanced removal by providing additional and more effective collector surfaces, such that the media bed was not truly “clean”; (b) high particle concentrations in the wastewater may have caused the ripening to occur quickly, such that the observed removals are actually postripening; (c) straining may have been a significant mechanism of particle deposition (Bradford et al. 2006; Herzig et al. 1970; Tufenkji et al. 2004), one that is unaccounted for in the clean bed model; and (d) heterogeneity in the media that resulted in media size and porosity gradients (as a function of depth) during media stratification at the end of backwashing, which could result in better than expected removal at the top of the media. Further, there are several factors that could cause the difference between observed and predicted values to decrease with increasing particles size, including (a) flocculation in the filter may have created a shift in the particle size distribution, creating the appearance of greater removal of smaller particles and subsequently less removal of larger particles; (b) greater detachment of larger particles; and (3) heterogeneity in the particle densities with size.

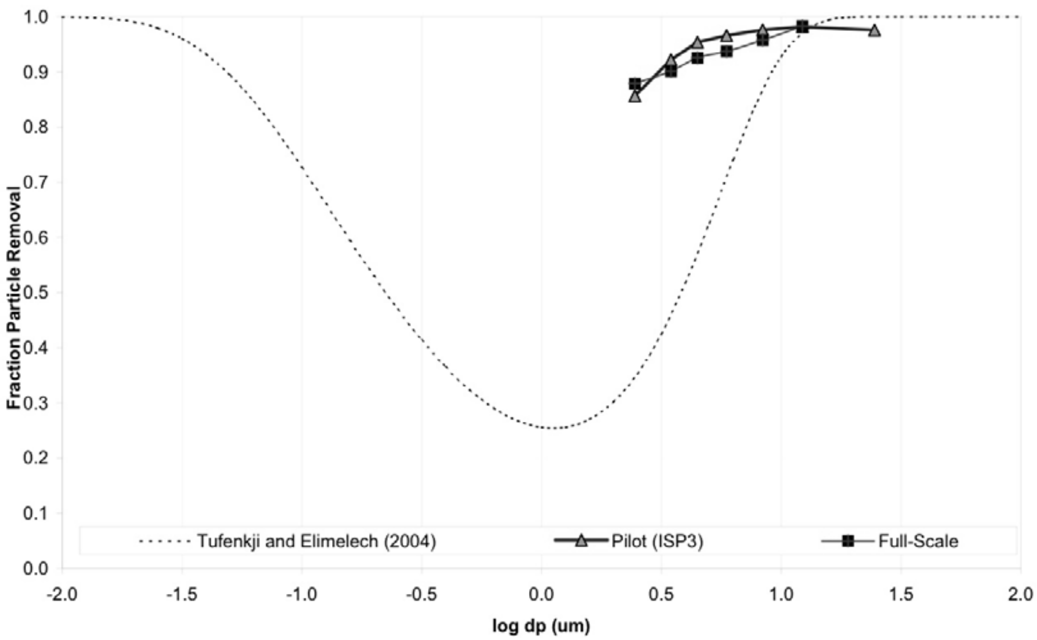

Figure 5.6. Comparison between observed clean bed removal during Phase I (pilot-scal = triangles) and Phase II (full-scale = squares) with predictions from theory (dashed line) at 5.0 gal/ft^2-min at Monterey.

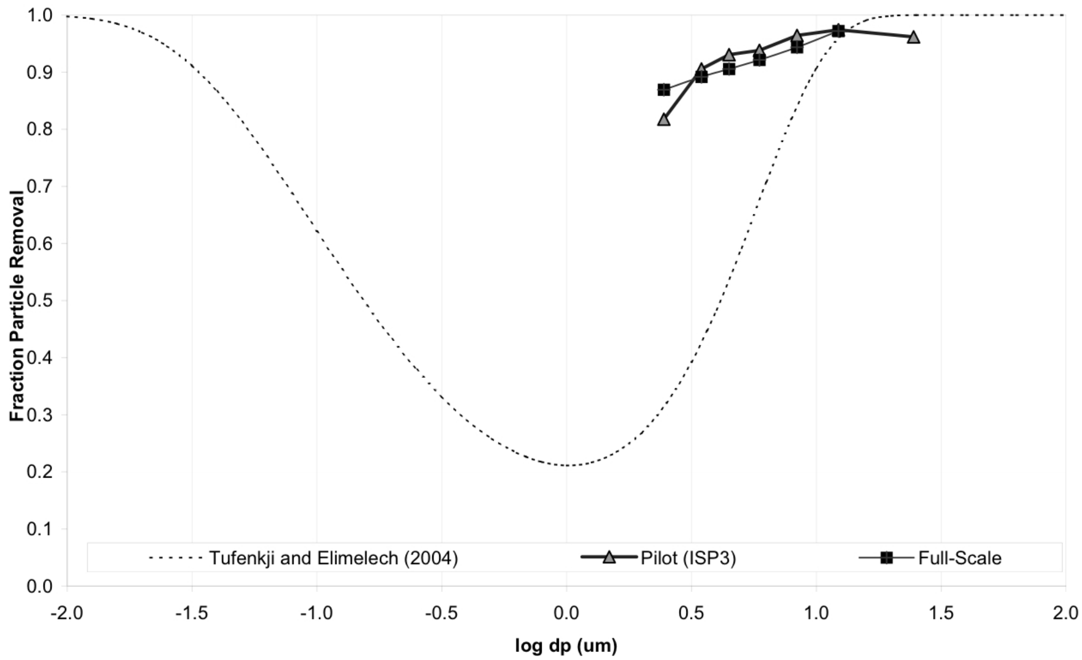

Figure 5.7. Comparison between observed clean bed removal during Phase I (pilot-scale = triangles) and Phase II (full-scale = squares) with predictions from theory (dashed line) at 7.5 gal/ft^2-min at Monterey.

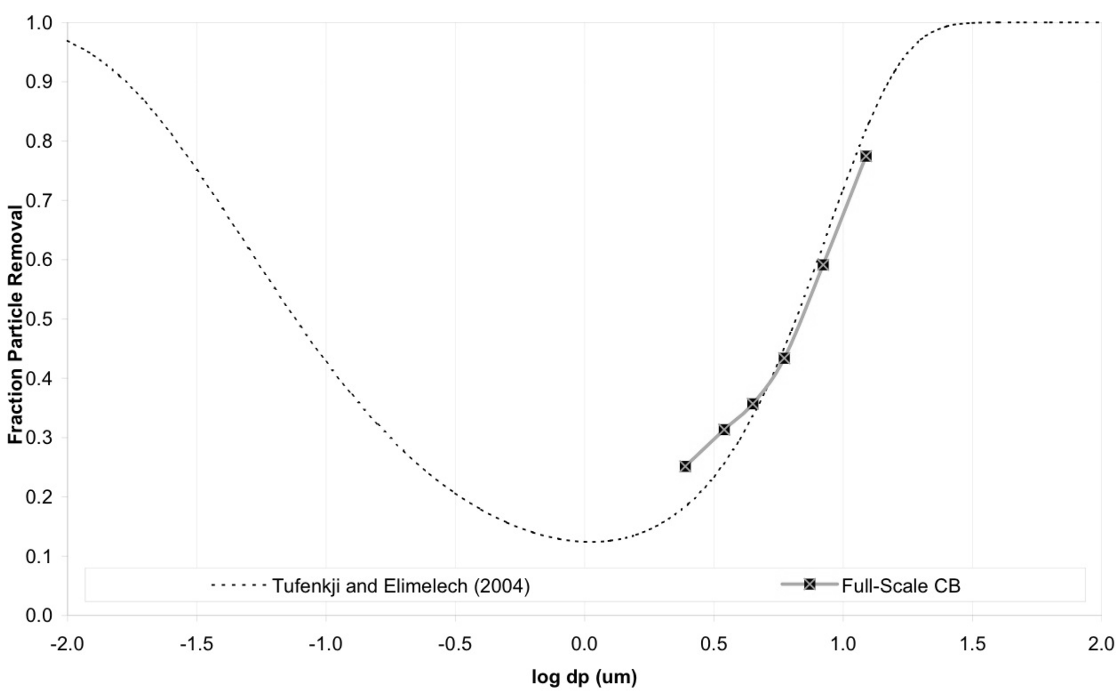

Figure 5.8. Comparison between observed clean bed removal during Phase II (circles) with predictions from theory (dashed line) at 5.0 gal/ft^2-min at San Jose.

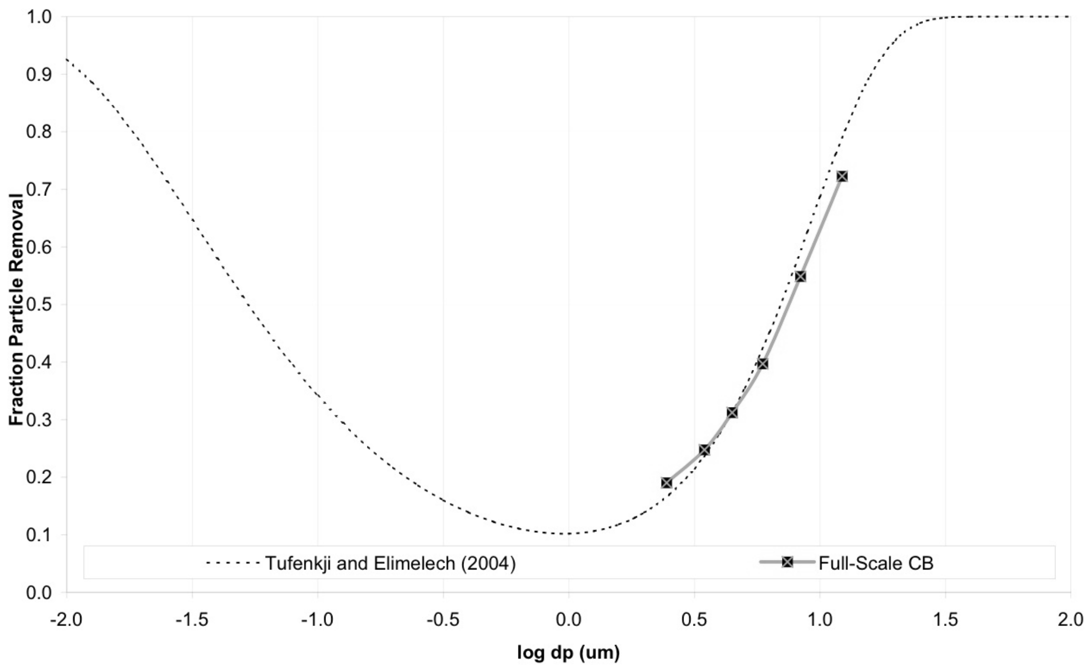

Figure 5.9. Comparison between observed clean bed removal during Phase II (circles) with predictions from theory (dashed line) at 7.5 gal/ft^2-min at San Jose.

CHAPTER 6

ROLE OF PARTICLE ASSOCIATION IN VIRUS REMOVAL BY COAGULATION AND FILTRATION

6.1 BACKGROUND

During the pilot-scale filtration runs (Chapter 3), the removal of MS2 was observed to be strongly dependent on the chemical (coagulant and polymer) dose. For example, very little removal of MS2 was observed during ISP1 (low polymer dose), whereas greater than 2-log removal was observed during ISP2 and ISP3 (higher polymer dose). Similarly, during the SRO runs, higher removal of MS2 occurred at the 7.5 gal/ft^2-min rate than at the 5 gal/ft^2-min rate. Another key observation from Phase I was that the removal efficiency of MS2 is expected to be dramatically affected depending on whether it is present as discrete virus particles or incorporated into wastewater particles. For example, if MS2 is incorporated into particles in the 1-μm size range, the removal efficiency is expected to decrease (compared to discrete virus particles), whereas if it is incorporated into particles 15 μm or larger, its removal is expected to increase.

These observations motivated a series of experiments to investigate the impact of coagulation on virus removal in more depth. The objectives of these experiments were (a) to quantify the particle association of bacteriophage with and without coagulant addition; and (b) to determine the impact of particle association on subsequent removal by filtration.

6.2 EXPERIMENTAL METHODS

Experiments were conducted at the pilot plant in Monterey. Three different bacteriophage were studied: MS2 coliphage, PRD1, and ΦX174. These phage were chosen based on differences in their size and net surface charge (see Table 6.1). The three phage were co-spiked into the pilot-plant influent wastewater. Two sets of filter runs were conducted, one in which coagulant was not added and one in which 30 mg/L JC1679 was used. The filters were operated at 5 gal/ft^2-min.

The particle association of the phage was measured in the pilot-plant influent (secondary effluent), flocculated influent, and filter effluents. Samples were processed by serial vacuum filtration through 12, 3, and 0.4-μm membrane filters. In each size fraction, phage were enumerated by standard plaque assay using the appropriate host bacterium (see Table 6.1).

Table 6.1. Characteristics of Bacteriophage MS2, PRD1, and ΦX174 (Dowd et al., 1998).

Bacteriophage	Isoelectric point	Diameter (nm)	Host bacterium
MS2	3.9	24	*E. coli* F_{amp} (ATCC 700891)
PRD1	4.2	63	Salmonella LT2 (ATCC 19585; care of M. Sobsey)
ΦX174	6.6	27	*E. coli* CN13 (ATCC 700609; care of M. Sobsey)

6.3 RESULTS FROM PILOT-SCALE EXPERIMENTS

The particle association of the three different phage is summarized in Figures 6.1, 6.2, and 6.3. In the absence of coagulation, the results were similar for all three phage: no significant particle association was observed and no significant removal by filtration occurred. With coagulant addition, however, interesting differences were observed. Whereas ΦX174 was not affected by coagulant addition (no particle association or removal by filtration), MS2 and PRD1 were affected. The measured concentration of MS2 decreased by about 1 log after coagulation in unfiltered samples. This decrease could be an artifact of the plaque assay, as multiple viruses in a single aggregate would still enumerate as a single plaque. However, experiments were also conducted in the laboratory with a jar test apparatus, and MS2 was enumerated by both plaque assay and quantitative, reverse-transcriptase polymerase chain reaction (qRT-PCR). The results using the qRT-PCR assay, which should not be affected by virus aggregation, also consistently showed a 1-log reduction that was due to coagulation alone (data not shown). These results suggest that MS2 may actually be inactivated by the coagulant (either the cationic polymer or the ACH).

Approximately 90% of the MS2 particles in the coagulated water were present in the 3–12 µm size fraction. According to the results in Figure 6.1, the MS2 particles in this size range were removed very efficiently (> 2-log removal). These results are consistent with the results based on particle counts presented in Figure 6.4. Most of the remaining MS2 particles were present in the 0.4–3 µm size fraction. Based on Figure 6.1, these particles were also removed efficiently, although high removal would not be expected based on the removal predicted by theory (see Figure 6.4). It is interesting to note that there appeared to be no net removal of MS2 in the size fraction < 0.4 µm.

The results for PRD1 were quite similar to MS2, with most PRD1 particles associated with the 0.4–12 µm size fractions, and high removal of these fractions by filtration. Thus, MS2 and PRD1 responded similarly, whereas ΦX174 was not affected by coagulation at all. These results suggest that surface charge may be an important factor influencing the particle association of viruses because of coagulation and their subsequent removal by filtration. The isoelectric point of ΦX174 has been determined to be 6.6, compared to 4.2 for PRD1 and 3.9 for MS2 (Dowd et al. 1998). The pH during the filtration runs was around 7.5. Thus, MS2 and PRD1 may have had greater negative surface charge than ΦX174, which resulted in more favorable interactions with the cationic polymer and/or the ACH flocs.

The results of these experiments have potentially important implications for virus removal by filtration. The findings suggest that removal is strongly dependent on preceding filtration with effective coagulation. Furthermore, effective coagulation is dependent on the surface characteristics of the virus. Coagulation may alter the size distribution of particles with which viruses are associated. Additional research is needed to investigate these mechanisms in more detail. In particular, data on pathogenic human viruses is needed. Also, a better understanding of the effect and interaction of different coagulants and polymers is needed.

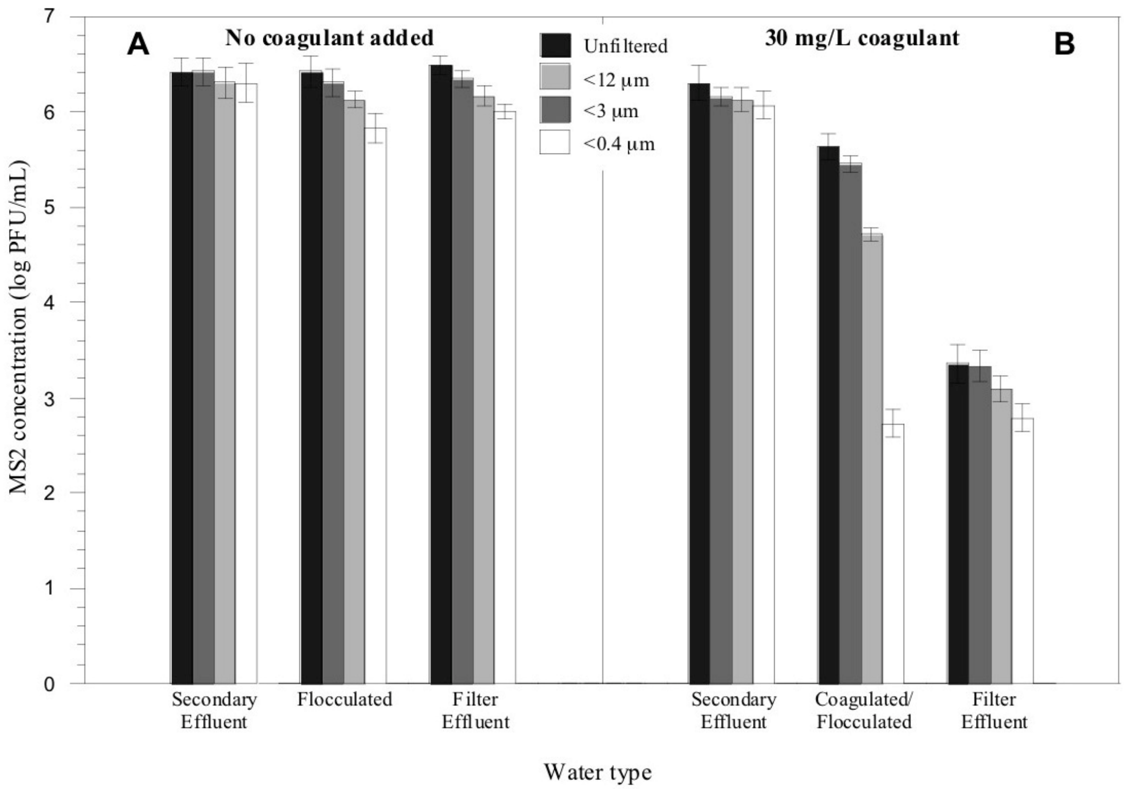

Figure 6.1. Particle association of MS2 in pilot-plant influent (secondary effluent), coagulated influent, and filter effluent.

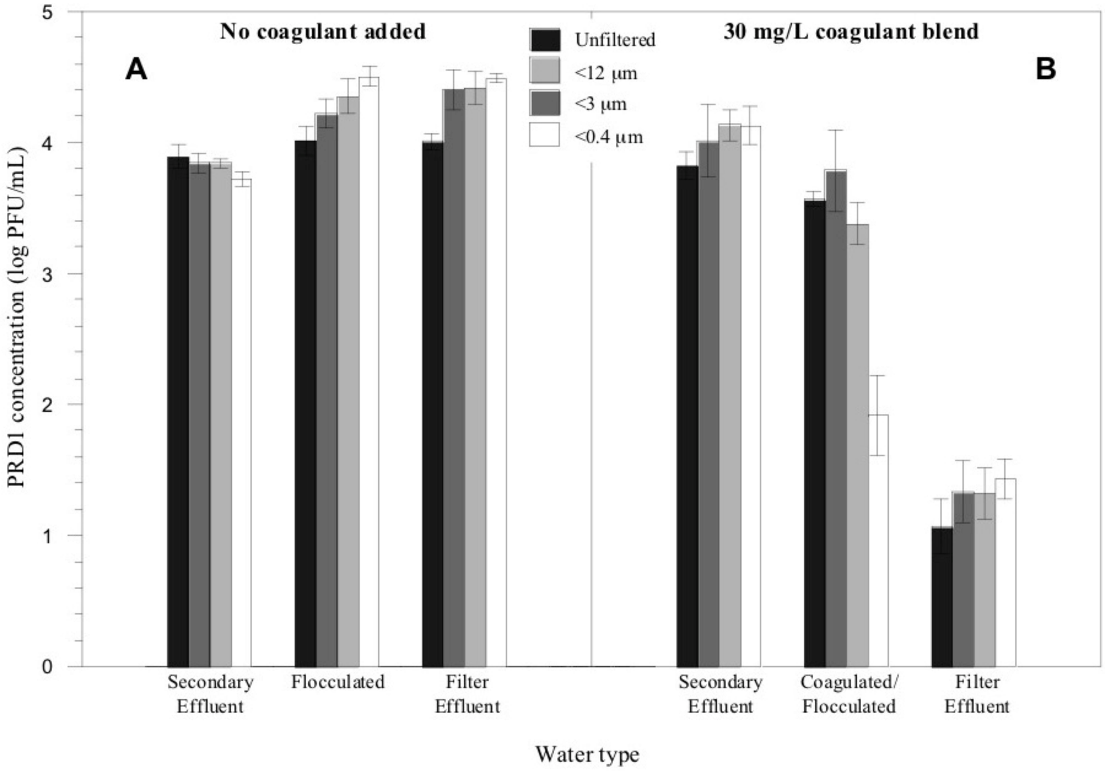

Figure 6.2. Particle association of PRD1 in pilot-plant influent (secondary effluent), coagulated influent, and filter effluent.

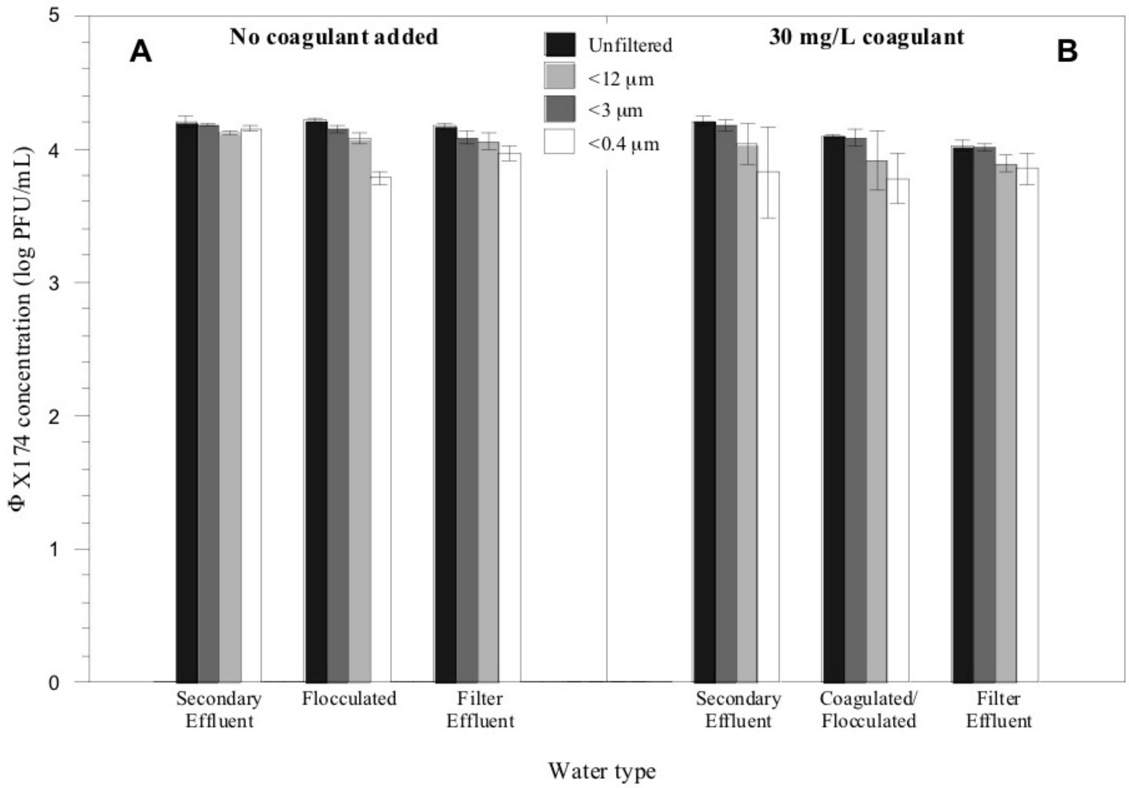

Figure 6.3. Particle association of ΦX174 in pilot-plant influent (secondary effluent), coagulated influent, and filter effluent.

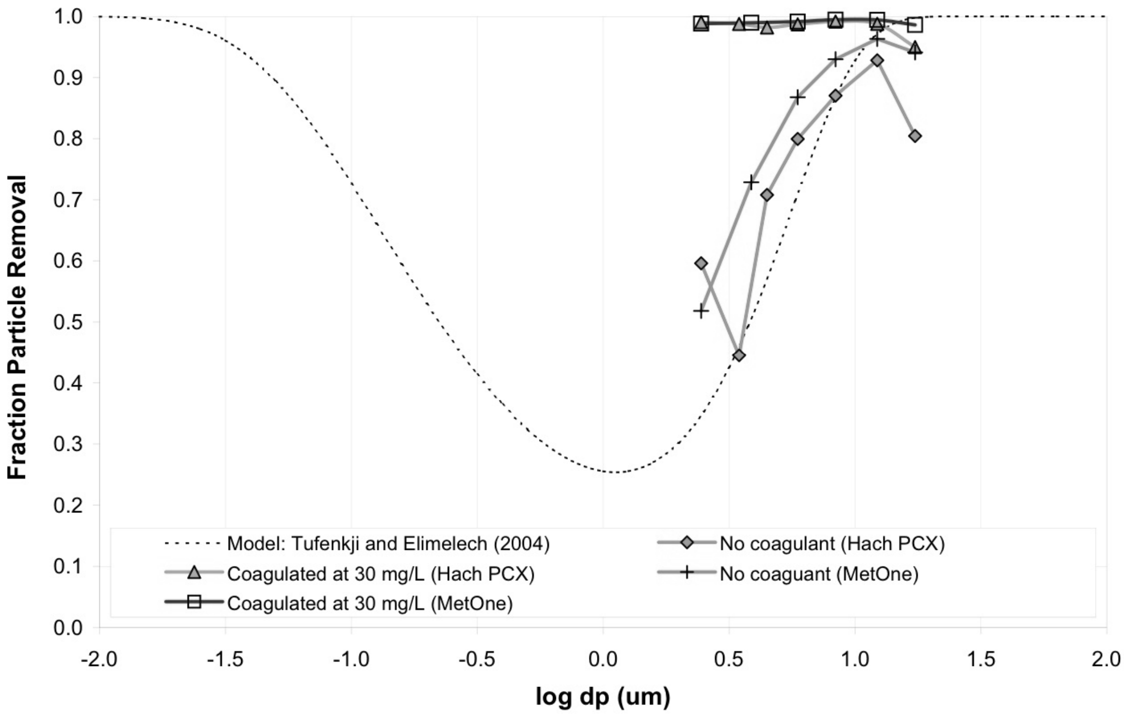

Figure 6.4. Role of coagulation in particle removal. Experiment with same influent water (water during pilot-scale virus–particle association test).

CHAPTER 7

CONCLUSIONS

7.1 SUMMARY AND CONCLUSIONS

More than 200 pilot filter runs were conducted during four test periods in Phase I. During the first three test periods, filters loaded at 5, 6.25, 7.5, 8.75, and 10 gal/ft^2-min received the same influent water (the coagulant dose was the same for all loading rates). The conclusions from these test periods are as follows:

- When the coagulant dose was the same for all loading rates, the removal efficiency of the filters decreased for all metrics (turbidity, particle counts in the 2–15 μm size range, total coliform bacteria, *E. coli*, and MS2 coliphage) as the loading rate increased.

- Although not explicitly investigated, it was observed that when filters performed well (i.e., high particle removal) the disparity between loading rates was greater. Likewise, when filters had low particle removal, the differences in filter performance between loading rates was minimal.

- Consistent with clean bed filtration theory, larger particles in the size range of 2–15 μm were removed more efficiently.

- The effect of loading rate on particle removal efficiency was consistent with filtration theory. However, the effect of loading rate was lower during clean bed treatment than later in the filter run.

- The apparent removal of MS2 coliphage by filtration alone was small (0.1 to 1 log), and was greater when higher coagulant and polymer doses were used. The apparent removal by coagulation plus filtration was much greater and increased with chemical dose (up to 3-log removal).

- Based on the head loss profiles in the filter bed, particles penetrated farther in the filter bed at higher filter loading rates.

- Minimal particle removal occurred through the sand layer (1 ft; 31 cm) compared to the anthracite layer (4 ft; 122 cm).

- The filter loading rate did not have a subsequent impact on the ability to disinfect the effluent with chlorine, even when higher loading rates had significantly lower particle removal.

During the fourth test period in Phase I, only two loading rates were tested: 5.0 and 7.5 gal/ft^2-min. The two loading rates were tested on an alternating basis (all five filters tested at the same rate and switching the rate was tested between runs) such that the coagulant dose could be continuously optimized for each loading rate to produce an effluent turbidity of 1.9 NTU (a similar turbidity target to the full-scale plant). The conclusions from this test period are as follows:

- Under these conditions, equivalent filter effluent quality was produced at both filter loading rates with respect to turbidity, particle counts, and removal of indicator bacteria.

- The average coagulant dose necessary to achieve equivalent performance was 62% higher at the 7.5 gal/ft^2-min rate (5.6 mg/L of coagulant, versus 3.5 mg/L at 5.0 gal/ft^2-min).

- The higher coagulant dose resulted in significantly higher removal of MS2 coliphage at the higher loading rate (1.58 versus 0.25 log removal for the 7.5 and 5.0 gal/ft^2-min rates, respectively).

- At the top of the filter bed, particle removal was greatest at the 5.0 gal/ft^2-min rate, but deeper in the media bed (below 6 in from top) particle removal was greater at the 7.5 gal/ft^2-min rate.

- The filter loading rate did not have a subsequent impact on the ability to disinfect the effluent with chlorine.

During Phase II, additional experiments were conducted at the pilot facility to better understand the role of coagulation and particle association on virus removal. Three different types of bacteriophage (MS2, PRD1, and ΦX174) were used in these experiments. Key conclusions are the following:

- Coagulation was necessary to achieve effective removal of phage by filtration.

- With coagulation, greater than 2-log removal of MS2 and PRD1 was observed, whereas insignificant removal of ΦX174 was observed. These differences are likely due to differences in the surface characteristics of the viruses.

- Viruses removed by filtration were primarily associated with particles in the 0.4–12 μm size range.

During Phase II, full-scale experiments were conducted at two facilities. Sixty-two filter runs were completed at the MRWPCA facility, and 40 runs were completed at the City of San Jose. At both facilities, equivalent effluent quality was produced at the two filter loading rates, as determined by criteria defined by CDPH.

Additional conclusions are as follows:

- To achieve equivalent performance at MRWPCA, the average coagulant dose was 51% higher when operating at 7.5 than when operating at 5.0 gal/ft^2-min (7.7 versus 5.1 mg/L, respectively). At San Jose, equivalent water quality was produced at both loading rates without the addition of a coagulant prior to filtration.

- At MRWPCA, the effect of loading rate on the removal efficiency was different for the three particle removal metrics. To produce equivalent effluent particle counts in the 5–15 μm size range at both rates, the coagulant dose had to be optimized such that the 2–5 μm particle counts and turbidity were actually lower at 7.5 gal/ft^2-min than at 5.0 gal/ft^2-min.

- At San Jose, the increases in turbidity, 2–5 μm particles, and 5–15 μm particles were 5%, 3.1%, and 11%, respectively, when the filter loading rate was 7.5 gal/ft^2•min compared to 5.0 gal/ft^2•min. None of these increases were statistically significant.

A key overall finding from the project was that the negative impact of increased filter loading rate on treatment performance was more apparent when effective coagulation was practiced prior to filtration. Thus, the impact of loading rate was greater when the removal efficiency of the filters was higher. At San Jose, it was not necessary to use coagulant to meet the 2 NTU standard because the influent wastewater (secondary effluent) already had low turbidity. In contrast, to achieve the required removals to meet the 2 NTU standard in Monterey, significant coagulant doses were necessary. The resulting impact of loading rate was minimal at San Jose, where no coagulant was used at either loading rate, but Monterey was required to use 51% higher coagulant dose to produce equivalent effluent quality at the higher loading rate.

The observed relationship between turbidity and particle counts was complex. The ratio of turbidity to particle counts in the secondary effluents was different at San Jose and Monterey. Furthermore, turbidity and particles were removed with different efficiencies from each other and were different at the two treatment facilities. As a result, the particle counts in the San Jose filter effluents were higher than the Monterey filter effluents, despite the lower turbidity at San Jose. Thus, a turbidity requirement of 2 NTU is not likely to result in similar particle counts and size distributions in filter effluents from different water recycling plants.

Most specific to this study, however, was the observation that the impact of loading rate on removal efficiency was not consistent for turbidity and particles. At the pilot plant, the decrease in removal efficiency of turbidity and particles as the loading rate increased was similar. However, at both full-scale facilities, as the loading rate increased the decrease in removal of larger particles (5–15 μm) was greater than for smaller (2–5 μm) particles and turbidity.

Clearly, coagulation and flocculation may influence the relationship between turbidity and particles, as these processes alter both the numbers of particles, as well as the particle size distribution. Unfortunately, the online instruments did not allow a complete characterization of the particle size distribution, as only particles in the 2–15 μm size range were measured. Given that the turbidity measurement is most sensitive to particles in the 0.1–1 μm size range, it is not surprising that the different particle metrics were not always correlated.

Despite its inability to mimic particle counts, turbidity is still recommended as the regulatory parameter for filter effluent quality. Particle counts are not recommended for several reasons:

- Online instruments only measure a small segment of the particle size distribution (2–15 μm).

- Online instruments are not currently reliable, as they cannot handle high particle counts present in some wastewaters, the data are highly variable, biological growth in the instrument tubing causes clogging, and accurate calibrations can be difficult.

- There is insufficient information to establish acceptable effluent particle counts that are protective of public health.

7.2 REGULATORY AND ECONOMIC SIGNIFICANCE

7.2.1 Regulatory Changes as a Result of this Study

A process was developed in this study for water reclamation plants in California to demonstrate that their tertiary granular media filters can be operated at a loading rate of 7.5 gal/ft^2-min with performance equivalent to that achieved when operating at 5.0 gal/ft^2-min. After obtaining a temporary waiver to conduct testing and then performing the actual testing, if a treatment facility successfully meets the equivalency criteria (see Section 4.2), a treatment facility can prepare and submit a final report to CDPH seeking permanent approval for operation at a higher loading rate. If CDPH concurs with the findings and approves a higher loading rate, a letter of support is sent to the Regional Water Quality Control Board (RWQCB). The RWQCB then can decide to issue a permanent waiver from the Title-22 filter loading rate and specify a new maximum loading rate.

Following this process, and using the results from the Phase II full-scale testing at the Monterey facility, MRWPCA requested and received a permanent waiver to operate their tertiary filters at a loading rate of 7.5 gal/ft^2-min. (A compilation of the correspondence resulting from the regulatory approval process is provided in Appendix E.) As a result, MRWPCA can now provide 50% more recycled water to farmers without building any additional filters. Thus, the results from this study already have changed water recycling practices in the State of California significantly. The results and procedures developed in this study for gaining approval for higher filter loading rates will help utilities meet the rapidly growing demand for recycled water, allow treatment facilities to maximize their recycled water production, and save Californians tens of millions of dollars.

7.2.2 Economic Impact of Filter Loading Rate

The relationship between maximum loading rate and capital cost of granular media filters can be approximated using a simple linear relationship, with cost directly increasing with filter surface area. The relationship between loading rate and operating and maintenance (O&M) costs is more complex, where costs will vary depending on the actual operating loading rate (i.e., plant flow), the treatment plant, and the nature of the expense. As loading rate increases, O&M costs will either (a) not significantly change (e.g., operator costs), (b) be proportional to the number of filters (e.g., filter maintenance and operating energy), or (c) be a function of the actual operating loading rate (e.g., coagulant costs and backwashing costs). Most treatment facilities are designed to accommodate growth in recycled water demand, where plant flows will increase over time, and thus, costs that are a function of the operating loading rate will also increase over time.

MRWPCA was originally designed with six filters in operation at a peak rate of 7.5 gal/ft^2-min. If the plant was limited to 5 gal/ft^2-min, three more filters would need to be constructed. A cost analysis was performed comparing these two scenarios at MRWPCA: (a) the plant is allowed to operate at loading rates up to 7.5 gal/ft^2-min and no additional filters are needed; and (b) the plant is limited to 5.0 gal/ft^2-min and three additional filters are built. In both scenarios, the same amount of water is recycled and the only differences are the maximum loading rate and the number of filters. The following assumptions were made in the cost analysis (see Table 7.1):

1. The number of days per year that recycled water is produced does not change (240 days; 8 months).[18]

2. To simplify the analysis, days were defined as either high flow days (within 10% of the maximum daily capacity) or non-high flow days (flow is less than 90% of maximum daily average capacity).

3. Annual growth in recycled water demand was estimated to be 2.1% per year, which affects both the average annual flow (increases from 18.8 to 29.6 mgd over 21 years) and the number of high flow days (increases from 100 to 240 days).

4. At times when the filters are operated above 5.0 gal/ft^2-min, 50% more coagulant is needed, where the 7.5 gal/ft^2-min scenario was estimated to exceed 5.0 gal/ft^2-min 50% of the time on high flow days and 0% of the time on non-high flow days.

5. At the 5.0 gal/ft^2-min rate, all of the filters are in operation on high flow days and only six filters are in operation on non-high flow days.

6. At 5.0 gal/ft^2-min, backwash frequency is once per day for each filter in service (i.e., 23-h run, followed by 1 h out of service) on both high and non-high flow days.

7. At the 7.5 gal/ft^2-min rate maximum, all filters are used on all days.

8. At 7.5 gal/ft^2-min maximum, filters are backwashed 1.3 times per day (i.e., 19 h filter run) on high-flow days and only once per day on non-high flow days.[19]

9. By 2030, the plant would undergo expansion and these factors would change (thus the analysis is only performed from 2009 through 2030).

Only factors that are expected to change with loading rate were considered (Table 7.2), and, thus, factors such as operator time and processes outside of the tertiary filters (with the exception of coagulant dose) were not considered in the costs. Unit cost information is estimated based on actual operating costs at the Monterey plant (see Table 7.2). The time value of money was assumed to be comparable to the increase in costs that are due to inflation over time, so no adjustments were needed to compare present and future dollars (i.e., the interest earned by saving money now is cancelled out by the increase in energy, material, and labor costs).

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[18] MRWPCA recycled water demand is seasonal as the water is used for agricultural irrigation.
[19] At maximum flow with all filters in service at with a maximum of 7.5 gal/ft^2-min, the average flow is 4.8 gal/ft^2-min. The 1.3 backwashes per day is estimated based on full-scale operations at an average loading rate of 5.0 gal/ft^2-min.

Table 7.1. Operating Assumptions Used in the Loading Rate Cost Comparison Between 5.0 and 7.5 gal/ft^2-min Operation at MRWPCA

Parameter	Units	Maximum Loading Rate (gal/ft^2-min)	
		5.0	7.5
Maximum average flow	mgd	29.6	29.6
Maximum peak flow	mgd	38.5	38.5
Number of filters in service		9	6
Total filter surface area	ft^2	6480	4320
Annual days of production (constant)	days	240	240
Annual growth in demand		2.1%	2.1%
Average daily flow in 2008	mgd	18.8	18.8
Average daily flow in 2030 (projected)	mgd	29.6	29.6
Number of high flow days in 2008	days	100	100
Number of high flow days in 2030 (projected)	days	240	240
Increase in coagulant usage when above 5.0 gal/ft^2-min		50%	50%
Fraction of time above 5.0 gal/ft^2-min on non-high flow days		0%	0%
Fraction of time above 5.0 gal/ft^2-min on high flow days		0%	50%
Backwash frequency (non-high flow)	per day	1	1
Backwash frequency (high flow)	per day	1	1.3

Table 7.2. Capital and O&M Cost for Granular Media Filters at MRWPCA

Expense Category	Units	Cost*
Capital cost	per filter	$1,730,000
Maintenance; labor	per filter per month	$187
Maintenance; parts and equipment	per filter per year	$2,000
Maintenance; media replacement	per filter per year	$917
Energy; filter operations (excluding backwash)	per filter per day	$15
Energy; filter backwashing	per filter per backwash	$100
Coagulant chemical costs	per million gallons treated <5.0 gal/ft^2-min	$30

Note. *All costs are in 2009 U.S. dollars.

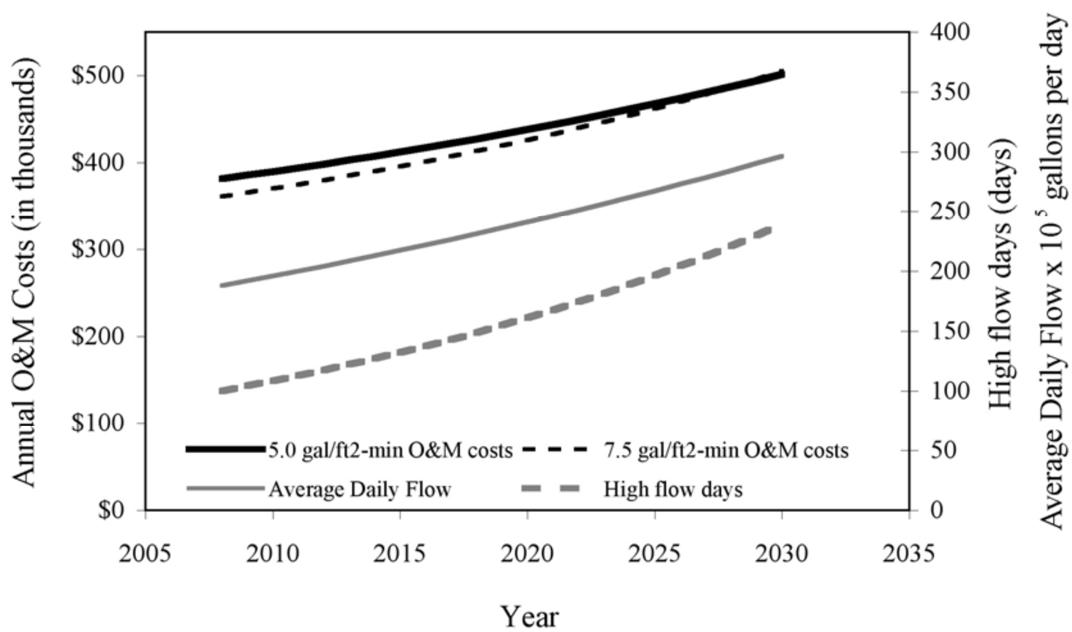


Figure 7.1. Projected O&M costs for operation at 5.0 versus 7.5 gal/ft^2-min with increases over time because of higher coagulated dose (a function of the average daily flow) and the energy required for backwashing (a function of the total number of high flow days).

The six Monterey filters were built between 1996 and 1997 for a cost of approximately $10.4 million[20] and the three additional filters that would be required if operations were limited to 5 gal/ft^2-min is estimated to be $5.2 million. The projected O&M costs are initially less for the higher loading rate (as there are fewer filters to maintain; see Table 7.3), but as the recycled water demand increases over time, the O&M costs for the two scenarios converge and by 2030, the 5 gal/ft^2-min scenario would be approximately the same (less than 1% difference; see Figure 7.1). The most significant O&M costs that will change with loading rate are (1) the cost of energy for backwashing (as the number of high flow days, and thus backwash frequency, increases) and (2) the cost of additional coagulant required when operating above 5 gal/ft^2-min (which also increases over time as the average plant flow increases; see Table 7.3 and Figure 7.1). As the average flow rate increases, the difference in coagulant costs between the two scenarios will increase ($15,000 versus $53,000 more in coagulant costs for the higher loading rate in 2009 versus 2030, respectively). The costs for maintenance and energy (including backwashing) decrease at the higher rate, because there are fewer filters to maintain. Adding everything together over the period considered for the cost analysis (2009 to 2030) the difference in O&M costs between the two scenarios is only about $300,000 saved by operating at the 7.5 gal/ft^2-min loading rate. Thus the total estimated savings as a result of operating with a 7.5 gal/ft^2-min maximum loading rate, including both capital improvements and O&M, is approximately $5.5 million for 2009 through 2030.

Table 7.3. Comparison of Total Capital and Projected O&M Costs for 2009 at the Monterey Plant When Operating With Maximum Filter Loading Rates of 5.0 and 7.5 gal/ft^2-min.

Expense Category	Units	Maximum Loading Rate (gal/ft^2-min) 5.0	7.5	Difference
Capital cost of additional filters	Total	$5,200,000	$0	-$5,200,000
Maintenance; labor	per year	$14,000	$9,000	-$5,000
Maintenance; parts + equipment	per year	$18,000	$12,000	-$6,000
Maintenance; media replacement	per year	$8,000	$5,000	-$3,000
Energy; filter operation (excluding backwash)	per year	$32,000	$22,000	-$10,000
Energy; filter backwashing	per year	$175,000	$164,000	-$11,000
Coagulant chemical costs	per year	$138,000	$153,000	$15,000
Total O&M costs in 2009	per year	$385,000	$365,000	-$20,000

[20] All costs are stated in February 2009 dollars using the 20-city construction cost index (CCI) consumer-price index adjustments (Engineering News Record 2009). “Construction Economics Report.” McGraw-Hill. Available online at http://enr.construction.com/economics

The costs savings will also be different at other treatment facilities. For the San Jose facility, where no operational changes were needed to produce equivalent water, the increased coagulant costs can be removed from these calculations and the cost savings would be even greater. The cost savings will also depend on how frequently the plant operates at higher loading rates. For example, in the winter, the City of Santa Rosa's Laguna Treatment Plant is required to filter its entire wastewater flow, which increases significantly during wet weather. Santa Rosa has already expanded filter capacity once during peak wet weather flow, but during large storm events, the filter capacity is still insufficient. For Santa Rosa, the increased filter capacity is only needed 1 to 3 days per year, so whatever increase in O&M costs that occur on these days would be trivial, compared to tens of millions of dollars that would be needed to increase the filter capacity to meet the observed peak flows.

7.3 AREAS FOR FUTURE RESEARCH

Several areas requiring further research were identified in this study, including investigating other ways to increase treatment plant capacity without capital improvements, developing a broader scientific base for other water recycling regulations, and improving our understanding of the role of particle-association in pathogen removal. In addition, the following specific research areas are recommended:

- **Higher loading rates**: The 18.3 m/h loading rate demonstrated in this study is not necessarily the upper limit on allowable loading rates for water recycling. The Monterey pilot- and full-scale filters experience significantly worse influent water quality (e.g., higher particle counts) than many tertiary filtration facilities. Higher loading rates certainly could be tested at the San Jose treatment plant; in particular, a significant improvement in turbidity or particle removal by filtration would be expected if coagulant was added prior to filtration, which might allow even greater increases in filter loading rate while still achieving adequate treatment. San Jose is well suited for testing additional filtration conditions, because it does not require regulatory approval and the bay discharge filters could be used as surrogates for the water recycling filters.

- **Free-chlorine versus chloramines:** The lower efficacy of chloramines compared to free chlorine for inactivation of viruses, in particular MS2, has been well documented (Sobsey 1989), but no distinction between the disinfection potential of free chlorine and combined chlorine is made in the California Water Recycling Criteria (apparently because at the time the regulations were written, nitrification was not well understood). The understanding and control of nitrification is much improved today, and it may be possible to demonstrate that free chlorine residual can be reliably used for disinfection at many water recycling treatment facilities. Because nitrification upsets can still occur, this sort of validation would be most appropriate for plants that have a secondary or alternative option in case of an upset, such as the inability to produce recycled water if nitrification fails. Research is needed to validate metrics and procedures that ensure complete nitrification and free-chlorine disinfection (e.g., online testing for free chlorine and ammonia). If such a study were successful in persuading CDPH to give credit for free chlorine disinfection, chlorine contact basins could be designed with much smaller footprints, significantly reducing the cost of recycled water.

- **Enhance virus filtration using coagulation:** Not all water recycling plants need to add coagulant prior to tertiary filtration, and many only add an absolute minimum to be in regulatory compliance (<< 1 mg/L). Even though these plants may be able to meet effluent turbidity requirements without effective coagulation and filtration, adequate coagulation may still provide significant benefits to public health protection by increasing virus removal through filtration. The effect of coagulation on virus removal through filtration should be investigated at plants with these low filter influent turbidities.

- **Improved filter performance metrics**: Turbidity, particle counts, total coliform bacteria, and seeded bacteriophage each provide a measure of filter performance, but each also has limitations as an indicator of granular media filter performance. In the full-scale study, determining which plant produced safer water depended on the metric used. Further development of more comprehensive filter performance metrics could provide tremendous benefit.

- **5-log virus removal:** The current California Water Recycling Criteria require 5-log reduction of virus to be demonstrated for alternative tertiary treatment processes, using seeded MS2 coliphage or poliovirus. This requirement should be revisited in light of the lower reductions achieved by currently approved processes (such as filtration followed by chloramination).

- **Additional virus-particle association testing:** In demonstrating the role of coagulant on virus removal through filtration, a 30 mg/L dose ACH and cationic polymer blend was used in all pilot-scale experiments. Low coagulant blend dose (5 mg/L) did not cause significant virus-particle association at the bench-scale, but coagulant blend doses between 5 and 30 mg/L should be tested to determine the efficacy of lower doses. In addition, experiments are needed to differentiate between the effect of polymer and coagulant on virus removal and particle association, as well as the relative effect of other doses and coagulants (e.g., ferric and alum). Further experiments on the ability to disinfect the filter effluent also are recommended to determine if the increase in virus-particle association that improved virus filterability would result in particle shielding during disinfection.

- **Study of indigenous phage or actual enteric organisms:** Indigenous phage and enteric viruses may behave differently than seeded phage. Studying native virus concentrations is difficult because of the low concentrations, but attempts to quantify the particle size distribution of indigenous virus-particle aggregates would be quite useful for understanding virus filtration. Tracking seasonal patterns and outbreaks of known enteric viruses, like rotavirus (Hejkal et al.,1984) and norovirus (da Silva et al., 2007), by surveying the medical community for viral outbreaks could be used to target sampling of wastewater when virus concentrations are expected to be higher.

REFERENCES

Adams, M. H. *Bacteriophages*; Interscience Publishers: New York, 1959.

Adin, A.; Elimelech, M. Particle filtration for wastewater irrigation. *Journal of Irrigation and Drainage Engineering-ASCE* **1989**, *115* (3), 474–487.

American Water Works Association (AWWA). *Operational Control of Coagulation and Filtration Processes*: Denver, CO, 1992.

American Water Works Association (AWWA). *Water Quality & Treatment*; McGraw-Hill Professional, New York, 1999.

American Water Works Association (AWWA), and American Society of Civil Engineers (ASCE). *Water Treatment Plant Design*; McGraw-Hill: New York, 1998.

Baumann, E. R.; Huang, J. Y. C. Granular filters for tertiary wastewater treatment. *Journal Water Pollution Control Federation* **1974**, *46* (8), 1958–1973.

Baylis, J. R. Seven years of high-rate filtration. *Journal American Water Works Association* **1956**, *48* (5), 585–596.

Bench, B. L.; Middlebrooks, E. J.; Dennis, B. G.; Reynolds, J. H. *Evaluation of Wastewater Filtration*; Utah Water Research Laboratory: Logan, UT, 1981.

Bradford, S. A.; Simunek, J.; Bettahar, M.; van Genuchten, M. T.; Yates, S. R. Significance of straining in colloid deposition: Evidence and implications. *Water Resources Research* **2006**, *42* (12).

Cal. Code Regs. §§ 60301–60475, Title 22, 2006.

Calendar, R. *The Bacteriophages*; Oxford University Press: New York, 2006.

Cleasby, J. L.; Baumann, E. R. Selection of sand filtration rates. *Journal American Water Works Association* **1962**, *54* (5), 579–602.

Clesceri, L. S.; Greenberg, A. E.; Eaton, A. D. *Standard Methods for the Examination of Water and Wastewater;* American Public Health Association/American Water Works Association/Water Environment Federation: Washington, DC, 1998.

Culp, R. L. Wastewater reclamation by tertiary treatment. *Journal Water Pollution Control Federation* **1963**, *35* (6), 799–806.

da Silva, A. K.; Le Saux, J. C.; Parnaudeau, S.; Pommepuy, M.; Elimelech, M.; Le Guyader, F. S. Evaluation of removal of noroviruses during wastewater treatment, using real-time reverse transcription-PCR: Different behaviors of genogroups I and II. *Applied and Environmental Microbiology* **2007**, *73* (24), 7891–7897.

Darby, J. L.; Lawler, D. F.; Wilshusen, T. P. Depth filtration of wastewater: Particle size and ripening. *Research Journal of the Water Pollution Control Federation* **1991**, *63* (3), 228–238.

Dawda, M. M.; Davidson, M. L.; Middlebrooks, E. J. Granular media filtration of secondary effluent. *Journal Water Pollution Control Federation* **1978**, *50* (9), 2143–2156.

Dowd, S. E.; Pillai, S. D.; Wang, S. Y.; Corapcioglu, M. Y. Delineating the specific influence of virus isoelectric point and size on virus adsorption and transport through sandy soils. *Applied and Environmental Microbiology* **1998,** *64* (2), 405–410.

Dugan, N. R.; Williams, D. J. Removal of *Cryptosporidium* by in-line filtration: Effects of coagulant type, filter loading rate and temperature. *Journal of Water Supply Research and Technology-Aqua* **2004,** *53* (1), 1–15.

FitzPatrick, J. A.; Swanson, C. L. *Evaluation of full-scale tertiary wastewater filters.* EPA-600/2-80-005; U.S. Environmental Protection Agency: Cincinnati, OH, 1980.

Fuller, G. W. *Report on the investigation into the purification of the Ohio River water at Louisville Kentucky*; D. Van Nostrand Company: New York, 1898.

Hejkal, T. W.; Smith, E. M.; Gerba, C. P. Seasonal occurrence of rotavirus in sewage. *Applied and Environmental Microbiology* **1984**, *47* (3), 588–590.

Herzig, J. P.; Leclerc, D. M.; Legoff, P. Flow of suspensions through porous media—Application to deep filtration. *Industrial and Engineering Chemistry* **1970,** *62* (5), 8–&.

Hutchison, W. R. High-rate direct filtration. *Journal American Water Works Association* **1976,** *68* (6), 292–298.

Jegatheesan, V.; Vigneswaran, S. Deep bed filtration: Mathematical models and observations. *Critical Reviews in Environmental Science and Technology* **2005**, *35* (6), 515–569.

Kau, S. M.; Lawler, D. F. Dynamics of deep-bed filtration: Velocity, depth, and media. *Journal of Environmental Engineering-ASCE* **1995**, *121* (12), 850–859.

Levenspiel, O. *Chemical Reaction Engineering*. John Wiley & Sons: New York, 1999.

Los Angeles County Sanitation District. *Pomona Virus Study Final Report*. Sanitation Districts of Los Angeles County. Prepared for California State Water Resources Control Board and United States Environmental Protection Agency, 1977.

Madigan, M. T.; Martinko, J. M.; Parker, J. *Brock Biology of Microorganisms*. Prentice-Hall: Upper Saddle River, NJ, 2000.

Mays, D. C.; Hunt, J. R. Hydrodynamic aspects of particle clogging in porous media. *Environmental Science & Technology* **2005**, *39* (2), 577–584.

Rajagopalan, R.; Tien, C. Trajectory analysis of deep-bed filtration with sphere-in-cell porous-media model. *AICHE Journal* **1976**, *22* (3), 523–533.

Rajala, R. L.; Pulkkanen, M.; Pessi, M.; Heinonen-Tanski, H. Removal of microbes from municipal wastewater effluent by rapid sand filtration and subsequent UV irradiation. *Water Science and Technology* **2003,** *47* (3), 157–162.

Sobsey, M. D. Inactivation of health-related microorganisms in water by disinfection processes. *Water Science and Technology* **1989**, *21* (3), 179–195.

Tchobanoglous, G. Filtration techniques in tertiary treatment. *Journal Water Pollution Control Federation* **1970**, *42* (4), 604–623.

Tchobanoglous, G.; Eliassen, R. Filtration of treated sewage. *Journal of the Sanitary Engineering Division* **1970,** *96*, 243–265.

Tebbutt, T. H. Y. Investigation into tertiary treatment by rapid filtration. *Water Research* **1971**, *5* (3), 81–92.

Tufenkji, N.; Elimelech, M. Correlation equation for predicting single-collector efficiency in physicochemical filtration in saturated porous media. *Environmental Science & Technology* **2004**, *38* (2), 529–536.

Tufenkji, N.; Miller, G. F.; Ryan, J. N.; Harvey, R. W.; Elimelech, M. Transport of *Cryptosporidium* oocysts in porous media: Role of straining and physicochemical filtration. *Environmental Science & Technology* **2004**, *38* (22), 5932–5938.

U.S. Environmental Protection Agency (EPA), and U.S. Agency for International Development (USAID). *Guidelines for Water Reuse*. EPA/625/R-04/108, Washington, DC, 2004.

Westerhoff, G. P. Experience with higher filtration rates. *Journal American Water Works Association* **1971**, *63* (6), 376–386.

Yakub, G. P.; Castric, D. A.; Stadterman-Knauer, K. L.; Tobin, M. J;, Blazina, M.; Heineman, T. N.; Yee, G. Y.; Frazier, L. Evaluation of Colilert and Enterolert defined substrate methodology for wastewater applications. *Water Environment Research* **2002,** *74* (2), 131–135.

Yao, K. M.; Habibian, M. M.; Omelia, C. R. Water and waste water filtration—Concepts and applications. *Environmental Science & Technology* **1971**, *5* (11), 1105–1112.

APPENDICES

APPENDIX A. PROTOCOLS FOR PILOT-PLANT OPERATION

1. CHEMICAL PREPARATION

Chemicals should be prepared fresh each day before system start-up. Chemicals should be prepared with reverse-osmosis (RO) water obtained from the MRWPCA laboratory or equivalent. Undiluted chemicals are obtained from the full-scale chemical containers at MRWPCA. Volumes of liquid should be measured using graduated cylinders (recommended 1000 mL graduated cylinder).

1-1: Coagulant ACH (Aluminum Chlorhydrate) dilution 1:4

- In a “2.5 gallon” (10L) sampling container add
  - 7.5 L of RO water
  - 2.5 L of coagulant ACH
- Mix well by vigorously shaking the container

1-2: Polymer (Cationic) Dilution 1:20

- In a “2.5 gallon” (10L) container add
  - 9.5 L of RO water
  - 500 mL of cationic polymer
- Mix well by vigorously shaking the container

2. SYSTEM START-UP

2-1: Ensure that following valves are closed:

- 2″ System influent bypass/drain valve (INFL BYPASS/DRAIN)
- 2″ Flocculation tank/channel drain valve at ground level (FLOC DRAIN)
- SCM sampling valve (SCM SAMPLE)
- Influent turbidity/particle counting sampling valve (INFL SAMPLE)
- All filters: 4″ backwash waste valves (D 1-1, D 2-1, D 3-1, D 4-1, and D 5-1)
- All filters: 1″ backwash drain valves (D 1-2, D 2-2, D 3-2, D 4-2, and D 5-2)
- All filters: backwash air line (AS 1, AS 2, AS 3, AS 4, and AS 5)
- All filters: backwash water line (BW 1, BW 2, BW 3, BW 4, and BW 5)
- All filters: filter effluent sampling valve (FESV 1, FESV 2, FESV 3, FESV 4, and FESV 5)
- All filters: electronic butterfly valve (Filter # 1 through #5 on SCADA)
- Backwash water influent line (HIGH FLOW and LOW FLOW)

2-2: Ensure that the following valves are open:

- 2″ System influent valve (SYSTEM INFL)
- ¾″ Influent valve next to influent flow meter (FMV INFL)
- 2″ Influent sampling drain valve (INFL SAMPLE DRAIN)
- All filters: 2″ filter effluent valve (FE 1, FE 2, FE 3, FE 4, and FE 5)
- All filters: ½″ filter effluent valve (FMV 1, FMV 2, FMV 3, FMV 4, and FMV 5)

2-3: Check that "class A" water hose is disconnected.

- Even though there is backflow protection, the "class A" hose should always be disconnected when operating the filter system.
- Check that it is disconnected from both influent sampling line hose bibb (HB INFL) and the system influent line hose bibb (HB SAMPLE).

2-4: Check that the FLEWR computer is running.

- Turn on SVRP laptop.
- Ctrl-Alt-Del User Name is Administration, Password is none (leave blank), Log onto SVRP Laptop (this computer).
- Check PLC connection (Ethernet cable – Blue CATSE 24 cable) cable connection to the SVRP laptop.
- Check mixer control cable connection (9 pin connector – Silver AWM cable) to the SVRP laptop.
- Load RS View Runtime
    - Open RS View Runtime (double click on Desktop Icon).
    - Select from the Menu: File → C:\REGIONAL\REGIONAL.RSV.
    - Click on "Load Project".
    - Click on "Filter Loading Project".
    - If alarm sound click on alarms, ACK ALL, close.

2-5: Set up chemical addition pumps.

- Check that fresh chemical dilutions have been prepared (Section 1: Chemical Preparations).
- For the polymer pump (center pump), place the open end of the peristaltic pump tube into polymer container.
- If the power to the pump is off, flip the black power switch on the face of the pump controller.
- Prime the pump to ensure that there is no tubing blockage.
    - Disconnect the peristaltic tubing from the FLEWR system connection.
    - Hold down the "PRIME" button on the controller face.
    - Wait until chemical pulses out of tubing.
    - If there is no flow after a minute, reverse the flow direction by pressing the "DIR" button, continue to hold the "PRIME" button until air bubbles are exiting the tubing into the chemical container, then set the direction back to normal by pressing the "DIR" button, and hold the "PRIME" button until there is chemical flow.
    - Re-connect the peristaltic tubing to the FLEWR system.

- Set desired flow rate
    - Press and hold the "FLOW" button until the display is flashing.
    - Use the up and down (▲, ▼) arrows to set display to the desired flow Set desired flow rate.
- Repeat this process for the coagulant pump (right most), and if applicable, the phage pump (left most pump).

2-6: Remove cap from pH meter.

- Loosen clamp on pH meter pipe.
- Remove pH meter from channel.
- Unscrew cap from end of pH meter.
- Place pH meter in channel two inches above the bottom.
- Tighten clamp on pipe.
- Leave screwdriver and cap on top of channel shelf.

2-7: Start the influent flow.

- In SCADA system, click on the tab "Setup" for the influent.
- Enter desired set point (**14.5**) and select "Write".
- Wait until the new set point is displayed.
- Click on "Start flow".
- Close the setup window by clicking on the "X".

2-8: Turn on chemical feed pumps.

- Press the "STOP/START" button to start chemical flows for the coagulant, polymer, and phage pumps.

2-9: Start the flocculation tank mixers.

- Wait until the flocculation tank is mostly full before starting the mixers.
- Open WinLink on the FLEWR computer (Start → Programs → WinLink → WinLink).
- Using the WinLink menu tabs load the user settings and start the mixers.
    - File → Load User Setup → "FLEWR1.set"
    - Mixers → Start
- Minimize WinLink program window by click on the "-"

2-10: Start flow to streaming current monitor (SCM).

- Wait until the flocculation tank and distribution channel are filled and there is overflow going to the drain, before starting flow to the SCM.
- Open the SCM valve no more than halfway (about 30° to 45°).
- Plug in the electrical cord for the streaming current sensor (lower gray box).

2-11: Start flow to influent sampling line.

- Open influent turbidity/particle counting sampling valve (INFL SAMPLE).

- Bleed air out of influent sampling line.
    - Open the bleed nozzle and drain air/water out of line for approximately 10 seconds into a bucket (if there is air in the line, you will hear it come out), then close the nozzle.
    - Pour waste into the trench drain (in the floor).
- Open and adjust turbidimeter valve until flow is 0.3 gpm (1.2 L/min).
- Check that there is a constant stream of water exiting PC waste.
    - If the flow is dripping out, try cleaning the PC with the brush and cleaning solution (5-3: Cleaning particle counters).
- Check that there is water flowing over PC weir.
- Check that the PC "Clean Sensor" light is off.

2-12: Wait 40 minutes for the water to run through the system.

2-13: Open effluent sampling line valves:

- Open and adjust the effluent sampling valves (FESV) until the electronic filter flow meters read approximately:
  0.57 gpm for the filter to be run at 2.5 gpm/sq. ft.
  0.63 gpm for the filter to be run at 5.0 gpm/sq. ft.
  0.67 gpm for filters to be run at 7.5, 10.0 or 12.5 gpm/sq. ft.
- Open and adjust the turbidimeter valve until flow is about 0.3 gpm (1.2 L/min). This step is best done together with the previous step because achieving the target flow rate requires the operator to go back-and-forth between the FESVs and the turbidimeter valves.
- Check that there is a constant stream of water exiting PC waste.
    - If the flow is dripping out, try cleaning the PC with the brush and cleaning solution (5-3: Cleaning particle counters).
- Check that there is water flowing over PC weir.
- Check that the PC "Clean Sensor" light is off.

2-14: Start filters.

- In SCADA, click on tab "Set up" for filter 1.
- Enter desired set point and select "Write".
- Wait until new set point is displayed.
- Click on "Start flow".
- Close the setup window by clicking on the "X".
- Repeat this for filters 2 through 5.

2-15: Periodically check the system.
- Check that the filter flow rate reaches and remains near the set point for all filters.
- Chemicals are flowing through pump tubing (2-5: Set up chemical addition pumps).
- Check the particle counters.
    - Clean sensor light is off.
    - A constant flow stream is exiting the particle counter (no dripping).
    - Water is flowing over the weir.

- Check that turbidimeter flow is above 0.2 gpm (0.8 L/min).
- If the PC clean sensor light is on, or waste flow is dripping, then try cleaning the sensor (5-3: cleaning particle counter). Note the event, date, and time in the log book.
- If there is no water flowing over a particle counter weir, slowly open the 3/4" filter effluent sampling valve corresponding to that PC, until there is overflow. Note the event, date, and time in the log book.
- If the flow to a turbidimeter is too low, adjust the turbidimeter flow valve and note the event, date, and time in the log book.

3. SYSTEM SHUTDOWN

The SCADA system will shutdown an individual filter for one of the following reasons: (1) terminal head loss has been reached, (2) the filter can no longer maintain the set flow rate, or (3) an operator has given the SCADA the shutdown command for that filter. Once all five filters have shutdown, manual shutdown of the remainder of the FLEWR system is required.

3-1: Check that all filters are shutdown/Operator filter shutdown.

- When it is time to shutdown the system, all filters must be shutdown. If some filters are still online (valve position is greater than 0%), the operator must send a command to the PLC to shutdown the running filters. Even though a filter is shutdown, flow will still go through the flow meter while the filter effluent sampling valve is open.
- To shut down an online filter:
    - In SCADA, click on tab "Set up" for the filters that is still running.
    - Click on "Stop flow".
    - Close the setup window by clicking on the "X".
    - The butterfly valve should close and the position should read 0%.
    - Repeat this for any other filters that are still running.

3-2: Turn off chemical feed pumps.

- Press the "STOP/START" button to start chemical flows for the coagulant, polymer, and phage pumps to stop the chemical additions.

3-4: Unplug streaming current sensor (lower gray box).

3-5: Close the following valves:

- SCM sampling valve
- Influent turbidity/particle counting sampling valve
- Influent turbidity/particle counting drain valve (after in sampling valve is closed)
- Filter effluent sampling valve for each filter
- Close the ½" filter effluent valve for each filter (before flow meter)

3-6: Using WinLink shutdown all mixers.

- Open WinLink on the FLEWR computer.
- Using the WinkLink menu tabs shutdown all mixers
  - o Mixers → Stop → Cancel
- Close the WinLink program.

3-7: Shutoff influent flow.

- In SCADA, click on tab "Set up" for the influent flow.
- Click on "Stop flow".
- Close the setup window by clicking on the "X".
- The butterfly valve should close and go to 0%.

3-8: Drain flocculation tank and distribution channel.

- Open drain valves:
  - o 2" flocculation tank/channel drain valve (ground level)
  - o All filters: 4" backwash drain valve
- Wait until both tanks and fully drained.
- Open the following valves to drain influent sampling piping:
  - o SCM sampling valve
  - o Influent turbidity/particle counting drain valve
  - o Influent turbidity/particle counting sampling valve
- After influent sampling piping is drained, close the following valves:
  - o SCM sampling valve
  - o Influent turbidity/particle counting sampling valve

3-9: Wash down tanks, mixer shafts, and pH probe (you must know how to operate the scissor lift and wear a hard hat).

- Use the hose on the scaffolding to wash down the tank walls and equipment, including the pH probe and mixer shafts. Be careful not to spray the mixer controllers or equipment.
- Lift the distribution channel weir, and wash all particles into the tank drain valves.
- Carefully curl up the hose and put it in a safe place that is not a tripping hazard.

3-10: Dispose of excess polymer and coagulant.

- Dump excess chemicals in FLEWR system trench drain (in the floor).
- Thoroughly rinse containers with tap water.

3-11: Backwash all filters (Section 4: Backwashing).

- All filters should be backwashed *as soon as possible* after system shutdown (within 12 hours).

3-12: Clean turbidimeters and particle counters.

- See Sections 5-1: Cleaning turbidimeters and 5-3: Cleaning particle counters.

3-13: Extended Shutdown

- If the FLEWR system will not be used for the next 5 days, see Section 6: Extended Shutdown.

4. BACKWASHING

AFTER all filters have shutdown and the system has been shutdown, each filter is then individually backwashed. BW= backwash

4-1: Drain filters

- Open 1" BW drain valve to drain the water level down to 12" above media.
- After draining, close 1" BW drain valve or you **COULD LOSE MEDIA.**

4-2: Ensure the following valves are CLOSED:

- Open 1" BW drain valve to drain the water level down to 12" above media.
- After draining, close 1" BW drain valve or you **COULD LOSE MEDIA.**
- All filters: 1" BW drain valve
- All filters: electric butterfly valve
- All filters: ½." filter effluent valve (before the flow meter)
- All filters: filter effluent sampling valve
- All filters: BW water valve
- All filters: BW air valve

4-3: Ensure the following valves are OPEN:

- All filters: 4" BW waste drain valve
- All filters: 2" filter effluent valve (at base of filter column)
- 2" Flocculation tank/channel drain valve (ground level)

4-4: Connect air compressor (Outside)

- Connect the compressor air supply line (red tube) to the FLEWR manifold (connect outside near wall).
- Plug in the compressor electric cord (outside).

4-5: Check backwash water valve flow rates

- Open BW water bypass valve (near trench drain).
- Open low rate BW water valve.
- Check that flow meter reading is 1.7 gpm; adjust flow controller if necessary.
- Close low rate BW water valve.
- Open high rate BW water valve.
- Check that flow meter reading is 11.6 gpm; adjust flow controller if necessary.
- Close high rate BW water valve.

4-6: Filter #1 Air scour – 4 minutes

- Open BW air valve and air scour the media for 1 minute.
- Open low rate BW water valve; open filter BW water valve; air scour with water for approximately 1 minute; WATCH the water/foam/media level rise; STOP the BW water (close the filter BW water valve) when the water/foam/media level reaches the filter column metal brace (between 1" BW drain valve and 4" BW waste drain valve). If you do not stop in time **you could lose media!**
- Continue to let the air scour run until that total air scour process has been 4 minutes.
- Close air scour valve and run low rate backwash water through the filter.

4-7: Filter #1 Backwash – 20 minutes

- Allow water level to slowly rise until there is overflow into the 4" BW waste drain valve.
- Simultaneously close low rate BW water valve and open the high rate BW water valve.
- Allow filter to backwash for 20 minutes, the media bed should fluidize, and rise about 12" above the normal (marked) level. Check that the high rate flow is still 11.6 gpm and staying constant. Watch that the media level does not get higher than 24" above the normal (marked) level.
- After 20 minutes, simultaneously open low rate BW water valve and close the high rate BW water valve.
- Close the 4" BW waste drain valve.
- Wait for the water level to rise and overflow into the distribution channel (check that the distribution channel drain is open).
- Close the filter BW water valve.
- Close the 2" filter effluent valve (at base of filter column).

4-8: Air scour and backwash Filters #2 through #5

- Repeat steps 4-4 and 4-5 for the remaining 4 filters.

4-9: Tap down media level

- With a rubber mallet, gently tap any and all of the media out of the 1" BW drain valve pipe for each filter.
- With a rubber mallet, gently tap the filter column (near the media), until the media level drops to the marked level.

5. ADDITIONAL EQUIPMENT MAINTENANCE

5-1: Cleaning Turbidimeters

See *Hach 1720D Low Range Process Turbidimeter Manual* pages 71–73.

- Turbidimeters should be cleaned daily while FLEWR system is in operation; cleaning should be done as part of the system shutdown.
- Clean the bubble trap wells with brush cleaner.
- Run clean water through turbidimeter.
    - For the influent turbidimeter, connect the class “A” hose line to the influent sampling line “bleed nozzle” (make sure that the influent sampling line drain is open).
    - For the filter effluent turbidimeters, run class “B” (backwash) water. Make sure all the 2" filter effluent valves are closed (at the base of the filter column), open the ½" filter effluent valve (before the flow meter) and filter backwash water valve and high rate backwash water valve. Slowly open the filter effluent sampling valve to allow flow to go to the sampling equipment.
- Remove the turbidimeter sensor head from the turbidimeter well, and place it safely out of the way.
- Drain turbidimeter well, by unplugging the drain plug from the bottom of the turbidimeter well, collecting the waste in a bucke.t
- Rinse turbidimeter well with clean water spray bottle, if necessary use a paper towel to clean off deposited material.
- Gently spray the turbidimeter sensor with isopropyl alcohol and if necessary clean with a kimwipe.
- Put the drain plug back into place.
- Put the turbidimeter sensor back into the turbidimeter well, and continue to run clean water through the turbidimeter until turbidity stabilizes.
- When all instrument cleanings are complete, shut off water to the turbidimeter/ particle counters .
    - Close the filter effluent sampling valve for all filters.
    - Close the filter backwash valves for all filters.
    - Close the high rate BW water valve.
- Note the cleaning event, date, and time in the log book.

5-2: Weekly Turbidimeter Calibration

See *Hach 1720D Low Range Process Turbidimeter Manual* pages 45–63 for calibration procedures.

- Turbidimeters should be calibrated before system start-up after an extended shutdown, and then weekly while the FLEWR system is in operation.
- Calibration can be performed with user-prepared standards, Stable-Cal standards, and can be checked with the ICE-PIC 20 NTU module (verification only).
- Stable-Cal standards will be for calibration for intensive sampling periods.
- User-prepared standards can be used for calibrations that are not during the intensive sampling periods.
- Note the calibration event, date and time in the log book.

5-3: Cleaning Particle Counters

See *Hach 2200 PCX Particle Counter Manual* page 43.

- Particle counters should be cleaned daily while FLEWR system is in operation.
- Disconnect the influent flow connector from the particle counter sensor (bottom of particle counter sensor).
- Allow water to drain from particle counter sensor.
- Dip the 2200 PCX particle counter cleaning brush in the particle counter cleaning solution.
- Insert cleaning brush into bottom fitting of the PC sensor, using a twirling motion, until you cannot insert the brush any farther.
- Repeat brush cleaning several times.
- Reconnect the influent flow connector to flush the sensor with clean water.
- Wait until a steady flow is exiting from the PC sensor (no dripping).
- When all instrument cleanings are complete, shut off water to the turbidimeter/ particle counters.
    - Close the filter effluent sampling valve for all filters.
    - Close the filter backwash valves for all filters.
    - Close the high rate BW water valve.
- Note the cleaning event, date, and time in the log book.

5-4: Check Particle Counter Flow Rate (100 mL/min)

See *Hach 2200 PCX Particle Counter Manual,* page 21.

- Clean particle counter. In addition to standard cleaning (see Section 5-3: Cleaning particle counters), clean the particle counter black tubing.
    - Unscrew the connection at the bottom of the weir and pull out the black PC influent tubing at the bottom of the weir.
    - Clean off the tubing with a paper towel.
    - Push the tubing back into the weir, screw the connection back into place.
- Run clean water through particle counter (follow procedure in Section 5-1: Run clean water through turbidimeter).
- Slide the particle counter sensor effluent cap away from the drain device.
- Use a 100 mL graduated cylinder to collect the water exiting the particle counter for exactly 60 seconds, the volume collected is the flow rate in mL/min.
- The flow rate should be 100 (99.5 to 100.5) mL/min.
- If flow rate is too low, raise weir system by 1" for every 1-2 mL below 100 mL
- If flow rate is too high, lower weir system by 1" for every 1-2 mL above 100 mL.

- Retest flow rate after weir height adjustment, readjust as necessary.
- Note the calibration event, date and time in the log book.

5-5: Particle Counter Calibration

- Particle counters should be calibrated at least annually. The date of the last calibration is posted on the right side of the counter.
- Calibration is performed by the Hach Company. Particle counters can either be sent back to Hach, or for the same price calibrated in-house by a Hach technician. Contact Cindy Lyver 800-227-4224 x3121 or the Northern California technician Geoff Harrison 800-227-4224 x2165.

5-6: pH Probe Calibration

- Clean and calibrate pH probe as specified by the manufacturer.

5-7: Streaming Current Monitor

- Clean and calibrate SCM as specified by the manufacturer.

6. EXTENDED SYSTEM SHUTDOWN (+5 Days)

After draining the tanks, backwashing all filters, and hosing the interior of the flocculation tank and distribution channel, you must perform the following tasks:

6-1: Drain Influent Line

- Close the 2" secondary effluent (class “C”) influent valve.
- Open the 2" influent bypass/drain valve.
- Check that 1" influent valve (between butterfly valve and flow meter) is open.
- In SCADA system
    - Click on tab "Setup" for the influent valve.
    - Select "Start Drain".
    - Allow water to drain from the influent piping.
    - Wait until flow through influent = 0 gpm.
    - Select "Stop Drain".

6-2: Particle Counter Shutdown

- First clean the particle counter as described in Section 5-3: Cleaning particle counters and run clean water through the sensor.
- Disconnect the influent flow connector (bottom of particle counter sensor).
- Spray isopropyl alcohol from the spray bottle into the particle counter sensor.
    - Gently push the nozzle of the alcohol spray bottle into the PC sensor waste line tubing.
    - Spray the isopropyl alcohol into the sensor for about 3 seconds.

6-3: Turbidimeter Shutdown

- Follow the procedure for cleaning the turbidimeter as described in Section 5-1: Cleaning turbidimeters, but leave the turbidimeter chamber empty after cleaning.

6-4: Setup Influent Bypass Line

- Check that influent butterfly valve is closed.
- Open the 2" influent bypass/drain valve.
- Open the 2" secondary effluent (class "C") influent valve about **half way.**

6-5: PLC Power Shutdown

After all equipment has been cleaned, the last task is to turn off the power to the sampling equipment.

- Inside the PLC cabinet, on the bottom left, locate the three circuit breaks.
- Leave the first breaker (left most) on; this one gives power to the PLC.
- Turn off (flip down) the 2nd and 3rd breakers.

7. START-UP AFTER EXTENDED SHUTDOWN (24 hrs before run)

If the power to the analytical equipment has been shut off, power must be restored for at least 24 hrs before data collection. It takes some time for the analog signals to warm up and give a steady signal to the PLC.

7-1: Turn on power to analytical equipment

- Inside the PLC cabinet, on the bottom left, locate the three circuit breaks.
- Make sure that the first breaker (left most) on; if it is off the PLC needs to be checked.
- Turn on (flip up) the 2nd and 3rd breakers; influent and effluent analytical equipment should turn on.

7-2: Insert particle counter "quick connect" sample line to sensor.

7-3: Recalibrate turbidimeters.

- The turbidimeters should be calibrated before the system start-up, and then weekly once the FLEWR system is in operation.
- See Section 5-2: Weekly turbidimeter calibration.

7-4: Perform any other necessary tasks in Section 5: Addition Equipment Maintenance

APPENDIX B. LIST OF PILOT-PLANT FILTER RUNS

Table B.1. Complete List of Pilot-Scale Filter Runs. ISP stands for “intensive sampling period” where the same coagulant dose was used for all loading rates. SRO stands for “single rate optimization” runs, where all filters were operated at the same loading rate and the coagulant dose was continuous optimized for that rate. F1 through F5 are the filter columns, where loading rate was varied by filter to remove any bias that is due to one individual filter.

Date		**Run**	**Start Time**	**Chemical**	**Dose (mg/L)**	**F1**	**F2**	**F3**	**F4**	**F5**
8/17/04	Tu	ISP1-Run 1	20:20	ACH/POLY	10/1.0	12.5	7.5	5	2.5	10
8/19/04	Th	ISP1-Run 2	18:39	ACH/POLY	15/1.0	5	10	12.5	7.5	2.5
8/24/04	Tu	ISP1-Run 3	11:55	ACH/POLY	15/1.0	12.5	10	5	2.5	7.5
8/26/04	Th	ISP1-Run 4	12:25	ACH/POLY	15/1.0	5	12.5	10	7.5	2.5
8/27/04	F	ISP1-Run 5	18:06	ACH/POLY	15/1.0	2.5	7.5	10	12.5	5
8/29/04	Su	ISP1-Run 6	15:14	ACH/POLY	15/1.0	10	7.5	2.5	12.5	5
8/31/04	Tu	ISP1-Run 7	11:20	ACH/POLY	15/1.5	2.5	12.5	7.5	5	10
9/1/04	W	ISP1-Run 8	16:37	ACH/POLY	15/1.0	7.5	5	2.5	10	12.5
9/3/04	F	ISP1-Run 9	14:15	ACH/POLY	15/0.0	10	2.5	12.5	5	7.5
9/4/04	Sa	ISP1-Run 10	21:41	ACH/POLY	15/1.0	12.5	5	7.5	2.5	10
9/6/04	M	ISP1-Run 11	15:29	ACH/POLY	15/1.0	7.5	2.5	5	10	12.5
9/23/04	Th	Coag. Dose 1	11:31	ACH/POLY	20/2	10	7.5	5	7.5	2.5
9/27/04	M	Coag. Dose 2	17:38	ACH/POLY	20/3	5	10	7.5	2.5	5
9/28/04	Tu	Coag. Dose 3	17:25	ACH/POLY	15/2	7.5	5	2.5	10	7.5
10/5/04	Tu	Full/Pilot Comp 1	8:02	JC1679	13	4.95	4.95	4.95	4.95	4.95
10/8/04	F	Coag. Dose 4	11:31	ACH/POLY	15/3	7.5	5	2.5	5	10
11/1/04	M	Coag. Dose 5	18:17	JC1679	10	7.5	10	10	5	7.5
11/3/04	W	Coag. Dose 6	19:23	JC1679	13	5	8.75	7.5	2.5	10
11/8/04	M	Coag. Dose 7	11:43	JC1679	15	7.5	10	8.75	6.25	5
2/4/05	F	Coag. Dose 8	9:57	JC1679	18	5	6.25	10	7.5	8.25

2/8/05	Tu	Coag. Dose 9	10:14	JC1679	26	7.5	8.75	5	6.25	10
2/11/05	F	Coag. Dose 10	11:55	JC1679	30	7.5	8.75	5	6.25	10
2/22/05	Tu	Coag. Dose 11	16:17	JC1679	40	10	7.5	8.75	5	6.25
2/24/05	Th	Coag. Dose 12	17:19	JC1679	60	5	6.25	7.5	10	8.75
3/2/05	W	Coag. Dose 13	11:45	JC1679	22	8.75	5	10	7.5	6.25
3/7/05	M	ISP2-Run 1	15:31	JC1679	30	10	6.25	8.75	7.5	5
3/9/05	W	ISP2-Run 2	13:57	JC1679	30	7.5	8.75	5	6.25	10
3/11/05	F	ISP2-Run 3	13:43	JC1679	22	8.75	5	7.5	10	6.25
3/14/05	M	ISP2-Run 4	14:44	JC1679	22	6.25	5	7.5	10	8.75
3/16/05	W	ISP2-Run 5	14:50	JC1679	30	5	10	8.75	6.25	7.5
3/18/05	F	ISP2-Run 6	14:17	JC1679	22	8.75	7.5	6.25	5	10
3/21/05	M	ISP2-Run 7	15:03	JC1679	22	10	8.75	5	7.5	6.25
3/23/05	W	ISP2-Run 8	14:50	JC1679	30	6.25	7.5	10	5	8.75
3/25/05	F	ISP2-Run 9	13:15	JC1679	18	5	6.25	10	8.75	7.5
3/28/05	M	ISP2-Run 10	13:02	JC1679	16	7.5	10	6.25	8.75	5
4/5/05	Tu	Backwash Test 1	12:20	JC1679	30	7.5	7.5	7.5	7.5	7.5
4/6/05	W	Backwash Test 2	13:13	JC1679	30	7.5	7.5	7.5	7.5	7.5
4/20/05	W	Backwash Test 3	12:30	JC1679	14	7.5	7.5	7.5	7.5	7.5

Table B.1 – Continued

Date		Run	Start Time	Chemical	Dose (mg/L)	F1	F2	F3	F4	F5
4/21/05	Th	Backwash Test 4	22:21	JC1679	14	7.5	7.5	7.5	7.5	7.5
4/27/05	W	Step Coag 1	9:13	JC1679	0-28	7.5	5	10	8.75	6.25
5/3/05	Tu	Full/Pilot Comp 2	8:00	JC1679	18-Nov	5	5	5	5	5
5/11/05	W	No Coag. Added	10:42	JC1679	0	10	6.25	7.5	5	8.75
5/18/05	W	Full/Pilot Comp 3	8:21	JC1679	13	5	5	5	5	5
5/20/05	F	Step Coag 2	9:45	JC1679	0-21	10	8.75	7.5	6.25	5
5/23/05	M	Step Coag 3	9:32	JC1679	0-21	7.5	6.25	5	8.75	10
5/25/05	W	ISP3-Run 1	12:46	JC1679	12	8.75	6.25	7.5	5	10
6/1/05	W	ISP3-Run 2	18:46	JC1679	13	10	7.5	5	8.75	6.25
6/7/05	Tu	ISP3-Run 3	13:58	JC1679	11	5	8.75	6.25	10	7.5
6/14/05	Tu	ISP3-Run 4	10:20	JC1679	9	10	7.5	5	6.25	8.75
6/16/05	Th	ISP3-Run 5	17:16	JC1679	11	7.5	5	8.75	6.25	10
6/22/05	W	ISP3-Run 6	18:56	JC1679	12	8.75	6.25	7.5	10	5
6/25/05	Sa	ISP3-Run 7	12:32	JC1679	12	6.25	10	8.75	7.5	5
6/28/05	Tu	ISP3-Run 8	10:25	JC1679	9	7.5	5	10	8.75	6.25
7/5/05	Tu	ISP3-Run 9	10:37	JC1679	10	5	8.75	6.25	10	7.5
7/7/05	Th	ISP3-Run 10	17:31	JC1679	11	6.25	8.75	10	5	7.5
7/11/05	M	ISP3-Run 11	12:01	JC1679	9.5	7.5	5	10	8.75	6.25
7/14/05	Tu	ISP3-Run 12	17:56	JC1679	10.3	5	10	6.25	7.5	8.75
7/22/05	F	Full/Pilot Comp 4	9:01	HyperIon 835	7-9	5	5	5	5	5
8/1/05	M	Step Coag 4	10:31	JC1679	0-14	6.25	10	5	8.75	7.5
8/8/05	M	Step Coag 5	16:08	JC1679	0-10	10	7.5	6.25	5	8.75
8/11/05	Th	SRO Test Run A	11:05	JC1679	5-10	7.5	7.5	7.5	7.5	7.5
8/16/05	Tu	SRO Test Run B	11:57	JC1679	4-8	5	5	5	5	5
8/22/05	M	ISP4-Run 1	11:57	JC1679	7.5	10	6.25	5	8.75	7.5

9/12/05	M	SRO- Run 1	12:26	JC1679	0.5-6.5	5	5	5	5	5
9/19/05	M	SRO- Run 2	12:40	JC1679	0.5-8.5	7.5	7.5	7.5	7.5	7.5
9/21/05	W	SRO- Run 3	12:45	JC1679	0.5-2.0	5	5	5	5	5
9/24/05	Sa	SRO- Run 4	12:55	JC1679	5-7	7.5	7.5	7.5	7.5	7.5
9/27/05	Tu	SRO- Run 5	11:07	JC1679	0.5-8.5	5	5	5	5	5
9/29/05	Th	SRO- Run 6	12:36	JC1679	0.5-8	7.5	7.5	7.5	7.5	7.5
10/1/05	Sa	SRO- Run 7	13:26	JC1679	0.5-7	5	5	5	5	5
10/3/05	M	SRO- Run 8	11:15	JC1679	4-8	7.5	7.5	7.5	7.5	7.5
10/5/05	W	SRO- Run 9	10:30	JC1679	0.5-5.5	5	5	5	5	5
10/6/05	Th	SRO- Run 10	17:36	JC1679	0.5-8	7.5	7.5	7.5	7.5	7.5

APPENDIX C. PROTOCOLS FROM FULL-SCALE TESTING AND VIRUS DISINFECTION

FLEWR Phase II Protocols: San Jose/Santa Clara WPCP

Study Purpose

The purpose of this study is to determine both the impact of higher loading rates on tertiary filter performance and the feasibility of operating the filters at such rates. A pilot scale study at the MRWPCA water reclamation facility tested loading rates from 2.5 to 12.5 gal/ft^2-min. Operation at filter rates 5 and 7.5 gal/ft^2-min produced equivalent water with average effluent turbidity <2 NTU, when coagulant/polymer doses were optimization specific to loading rate. Based on these pilot results, the CDPH is allowing full-scale loading rate testing, on a plant-by-plant basis, at treatment plants participating in Filter Loading Rate Evaluation for Water Reuse (FLEWR) Phase II study. This Phase II study will compare operation at 5 and 7.5 gal/ft^2-min using actual tertiary filters producing "Title-22" reclaimed water. Various metrics will be used to evaluate filter performance, including:

- Turbidity removal
- Particle removal (2–20 μm)
- Removal of coliform organisms
- Filter effluent disinfectability of coliform bacteria

In addition, feasibility of higher rates will be evaluate using the following operational data:

- Filter run length
- Filter production capacity
- Coagulant doses
- Clean bed head loss and changes backwash requirements

There are two basic filter testing configurations for this study: (1) simultaneous testing, where two filters receive the same influent water but are operated at different loading rates; or (2) consecutive testing, where only one test filter is used and the test loading rate is switched after some number of filter runs. Filter effluent disinfectability is monitored through either the full-scale disinfection system or through laboratory batch testing. The operational protocols are specific to the treatment plant and are dependent on the filter loading rate control scheme, secondary effluent quality, and other operational requirements. The following protocols describe the requirements for filter operations, instrumentation, data recording, laboratory analysis, and data reporting for the San Jose/Santa Clara Water Pollution Control Plant (SJSC WPCP).

Treatment Plant Background

SJSC WPCP is a tertiary wastewater treatment plant with 167 million gallons per day (MGD) design capacity and 270 MGD peak capacity. On average in 2006, the plant operated at an average flow rate of 122 MGD, with a daily minimum of 81 MGD and daily maximum of 176 MGD. The treatment process employs screening and grit removal, primary sedimentation, secondary Biological Nutrient Removal process, tertiary filtration, and chlorine disinfection processes. The plant influent is comprised of roughly 90% domestic sewage, and 10% industrial wastewater that have been pretreated. In 2006, the plant produced on average 9.0 MGD of Title 22 quality recycled water with a daily minimum of 2.8 MGD and daily maximum of 19.6 MGD.

The plant has four banks of filters, 16 dual media (sand and anthracite) filters divided into two batteries, of which 4 filters (B5–B8) are dedicated to Title 22 recycled water production with loading rates not to exceed 5 gpm/ft^2. The remaining 12 filters (A1–A8 and B1–B4) are used for the tertiary treatment (non–Title-22) of the plant final effluent prior to bay discharge. The non–Title 22 filters are identical in design to the Title-22 filters and are not limited to the 5 gpm/ft^2 regulatory limit. In agreement with CDPH, it was decided that non–Title-22 filters would be used to test the FLEWR Phase II objectives for SJSC WPCP, providing the benefits of avoiding any impacts to the recycled water production, and eliminating the need for a Phase II testing waiver from CDHS.

Filter Operation

Operating Configuration

Phase II at SJSC WPCP will follow the simultaneous testing mode, where two full-scale filters (Filters B3 & B4) are designated as the test filters, and the test loading rate will alternate between filters every run. The loading rate on the testing filter is held at a constant set point for the entire filter run, until the filter is shutdown. All other non-testing filters are unaffected by the study and will be operated to accommodate the fluctuations in plant flow.

Test Filter Start-Up

After filter shutdown and backwashing, the test filter is restarted with the following conditions:

- The test loading rate will be switched (either to 5 or 7.5 gpm/ft^2) from the previous run's loading rate.
- Immediately after reaching the target set point in the test filter, the operator notes the filter head loss value, noted as CBHL (clean bed head loss) in the log.
- Filters will be restarted shortly after backwashing. Even though diurnal variation in secondary effluent quality are minimal, the start times will be randomized by variation in the previous run's length to eliminate potential bias.

Coagulation Control Strategy

Currently, alum is only added if secondary effluent turbidity is greater than 5 NTU. In such a case, only a minute of alum is added (<< 1 ppm). The coagulant dose is not calculated and such calculations cannot be calculated in the DCU. If alum addition is needed, the DCU will only log that it was added (but not the dose). If coagulant addition is needed to achieve equivalent filter performance, then further investigation will be made into determining the coagulant dose.

Test Filter Shutdown Criteria

The filter is shutdown and backwashed after reaching maximum head loss. After the filter flow has stopped, the test filter is not restarted until the filter is backwashed.

Testing Period

- *Preliminary data collection*: A preliminary data set of 10 filter runs will be initially collected. The FLEWR team will analyze this data and provide feedback prior to further testing.
- *Required number of runs*: Based on the preliminary data set, the FLEWR team will estimate the minimum number of testing runs required at each treatment plant to achieve the statistical requirements of the study.

Monitoring for Backwash Changes and Other Operational Changes

- *Monitoring backwash requirements*: Although no changes in backwashing requirements are expected, it is necessary to monitor backwash effectiveness, and if needed, to make changes to the backwashing procedure. A statistically significant increasing trend in the CBHL is used as an indicator that backwashing changes are needed. Record information on the backwashing procedure and note any procedural changes.
- *Other operational changes*: It is important to avoid any unnecessary process changes during the testing period. If changes to the tertiary system are needed please discuss with the FLEWR team before making process changes. Also, please note and report any abnormalities and/or plant maintenance activities that impacted the secondary effluent water quality or tertiary treatment.
- *Suspension of FLEWR testing*: If for any reason Phase II testing must be suspended, stop Phase II testing and resume normal/necessary plant operations. When able, please notify the FLEWR team.

<u>Instrumentation</u>

Required Online Instrumentation

- *Turbiditimeter*: Hach 1720D or 1720E or equivalent
  - Flow rate though the turbidimeter is approximately 750 mL/min
- *Particle Counter*: Hach PCX 2200 or equivalent
  - Set with continuous size ranges >2, >3, >4, >5, >7, >10, >15, >20 μm
  - Instrument flow rate should be 100 mL/min
  - Particle counter dilution system on the secondary effluent sampling line, if total particles (>2 μm) is greater than 16,000 particles/mL

Instrument Locations

The instrumentation is configured to monitor water quality in the following locations:

- *Secondary effluent*: Particle counter (no dilution needed) and turbidimeter
- *Test filter effluent B3 and B4*: Particle counter (no dilution needed) and turbidimeter

Instrument Calibration and Cleaning

Proper instrument calibration is required. When feasible, ALL CLEANING AND CALIBRATIONS should be completed WHEN THE TEST FILTER IS SHUTDOWN.

- Turbidimeters
  - Calibrated prior to testing period using HACH Formazin standards of 0 and 20 NTU.
  - Turbidimeters are cleaned weekly.
- Particle counters
  - Annual calibrations are performed by Hach factory technician.
  - The y-strainer for the secondary effluent particle counter influent line is cleaned twice daily.

- The secondary effluent particle counter flow rate is checked daily (flow should be 100 mL/min); adjustments are made if flow is *not* between 98–02 mL/min.
- All particle counters are cleaned weekly, and flow rates are checked after each cleaning.

Data Collection

DCU Data Collection

The DCU data will be recorded every one minute. These points will be average values for the one-minute period. The data collection system is configured for this recording frequency.

The following data will be collected by the DCU:

Influent parameters

- Date and time
- Filter influent turbidity (NTU)
- Filter influent particle counts (>2, 3, 4, 5, 7, 10, 15, and 20 μm) (particles/mL)
- Alum addition (whether it is being added or not)
- pH of filter influent

Parameters for each filter

- Date and time
- Run time (h)
- Loading rate (gal/ft^2-min)
- Cumulative volume treated (gal/ft^2)
- Head loss (ft)
- Filter effluent turbidity (NTU)
- Filter effluent particle counts (>2, 3, 4, 5, 7, 10, 15, and 20 μm) (particles/mL)

Operating Log

The operator will record the following data parameters in the FLEWR Phase II operator sheet at the completion of each run:

- Filter operated at which rate
- Start-up and shutdown times
- Run length (h)
- Clean bed head loss (ft); head loss at filter start-up following a backwash; once the loading rate has reach the target value
- Terminal head loss (ft)
- Backwash length (min)
- Backwash volume (gal/ft^2)
- Any notes regarding operation, such as problems encountered or anything out of the ordinary.

Laboratory Analysis

Grab Sampling

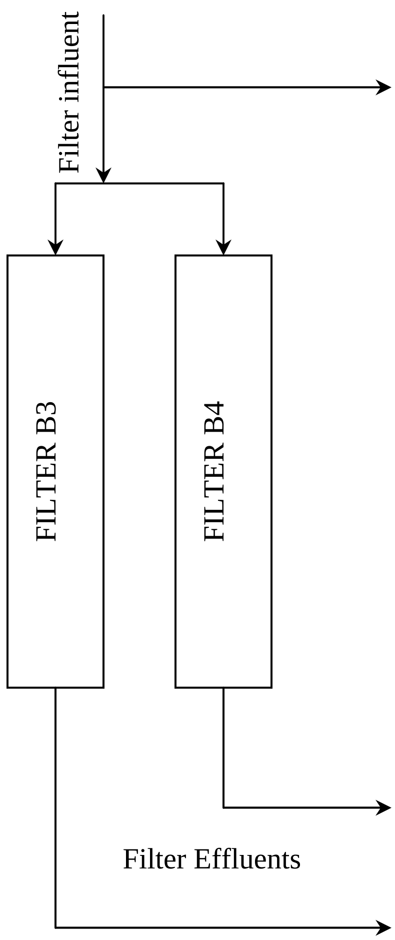
Grab samples of both filter influent and effluent (both filters) will be taken three times during each run according to the "SJSC WPCP FLEWR Sampling Flow Chart" that follows. Filter sample collection times are based on times when filter head loss is at: 2.0 feet, 5.5 feet and 9.5 feet. This sampling time schedule approximates the initial, mid, and end run samples during each filter run.

Total coliform and *E. coli* will be measured on all samples, and disinfection experiments will be conducted on the effluent samples. Plant operators will collect three 200mL samples of filter influent and six 2-L samples of filter effluents for each filter run (which is roughly a day) and measure the pH and temperature at time of sample collect. Operators will deliver samples to the plant lab for fecal and coliform analysis.

If any unforeseen circumstances occur (i.e., storm flow events, upstream process upsets, power failures, mechanical problems, telemetry problems, etc.) that might affect the scheduled filter run, the filter operator should document the problems in the FLEWR log book and in the FLEWR filter operating data matrix and schedule sheet.

SJSC WPCP FLEWR Sampling Flow Chart

- For each filter run, there will be **3 sampling periods** spread over the run time.
- For **each** sampling period, the following samples should be collected:



Filter Influent Sample

- 200 mL sample collected
- Measure pH and temp at sample collection
- Total coliform/*E. coli* analysis

Filter Effluent Samples (both filters)

- 2-L sample collected
- Measure pH and temp at sample collection
- Total coliform/*E. coli* analysis (pre-chlorinated)
- Bench-scale disinfection experiment

Total number of total coliform/*E. coli* samples per run		
1 filter influent x 3 sample times/run	=	3 samples/run
2 filter effluents x 3 sample times/run	=	6 samples/run
2 chloramine disinfected x 3 sample time/run	=	6 samples/run
2 chlorine disinfected x 3 sample times/run	=	6 samples/run
Total number of samples	=	21 samples/run

Coliform Analysis

The laboratory tests for the presence of total coliform bacteria and *Escherichia coli* in the treatment plant filter influent and filter effluent samples uses the Enzyme Substrate Test method, Standard Method 9223B. For this project Colilert-18 method is recommended.

Laboratory Protocol

Sampling Procedures

- Microbiology lab will supply OPS with autoclaved and sterilized bottles without sodium thiosulfate as requested.
- Collect all filter influent samples in sterile 200 m L plastic coliform bottles.
- Collect all filter effluent samples (B3 and B4 filters) in sterile 2-L wide-mouth plastic bottles.
- When the sample is collected, leave ample air space in the bottle (at least 2.5 cm) to facilitate mixing by shaking, before examination. Collect samples that are representative of the water being tested, flush or disinfect sample ports, and use aseptic techniques to avoid sample contamination.
- Keep sampling bottle closed until it is to be filled.
- The samples should be delivered to the lab refrigerator as soon as possible.

Testing Procedure (SM 9223B)

1. Carefully separate one Snap Pack from the strip taking care not to accidentally open adjacent pack.
2. Tap the Snap Pack to ensure that all of the Colilert powder is in the bottom part of the pack.
3. Open one pack by snapping back the top at the scoreline.
4. Add the reagent to the 100 ml water sample in a sterile, transparent, non-fluorescent vessel.
5. Aseptically cap and seal the vessel.
6. Shake until dissolved.
7. Pour sample in a sterile Quanti-tray pack.
8. Incubate for 18 hours at 35 ± 0.5° C.
9. Read the results at 18 hours. Compare each result against the comparator dispensed into an identical vessel.

Results Interpretation

1. If no yellow color is observed, the test is negative.
2. If the sample has a yellow color equal to or greater than the comparator, the presence of total coliforms is confirmed. If color is not uniform, mix by inversion and then recheck.
3. If sample is yellow, but lighter than the comparator, it may be incubated an additional 4 hours (But no more than 22 hours total). If the sample is coliform positive, the color will intensify. If it does not intensify, the sample is negative. Any sample showing the yellow color after incubation periods longer than 22 hours is not valid.
4. If yellow is observed, check vessel for fluorescence by placing a 6 watt 365 nm UV light within five inches of the sample in a dark environment. Be sure the light is facing away from your eyes and toward the vessel. If fluorescence is greater or equal to the fluorescence of the comparator, the presence of *E. coli* is confirmed.

Procedural Notes

1. If an inoculated colilert sample is inadvertently incubated for more than 22 hours, the following guidelines apply: Lack of yellow color is a VALID NEGATIVE TEST. A yellow color after 22 hours is not valid and should be repeated or verified.

2. If water sample has a background color, compare inoculated colilert sample to a control blank of the same water sample.

3. Use sterile water not buffered water for making dilutions.

Quality Control

Inoculate sterile water containing colilert media with the following cultures:

Positive Controls:	*Expected Result*
E. coli	Yellow, fluorescent (MUG-positive)
Klebsiella pneumoniae	Yellow, no fluorescence (MUG-negative)

Negative Control:	*Expected Result*
Pseudomonas auriginosa	Clear, no fluorescence (noncoliform)

Sample Invalidation

1. Indeterminate color at 22 hours invalidates a sample.
2. The time from sample collection to initiation of analysis must not exceed 8 hrs.
3. The minimum sample volume allowed is 100 mls.
4. Samples must not be frozen.
5. Samples must be transported at < 10 °C.

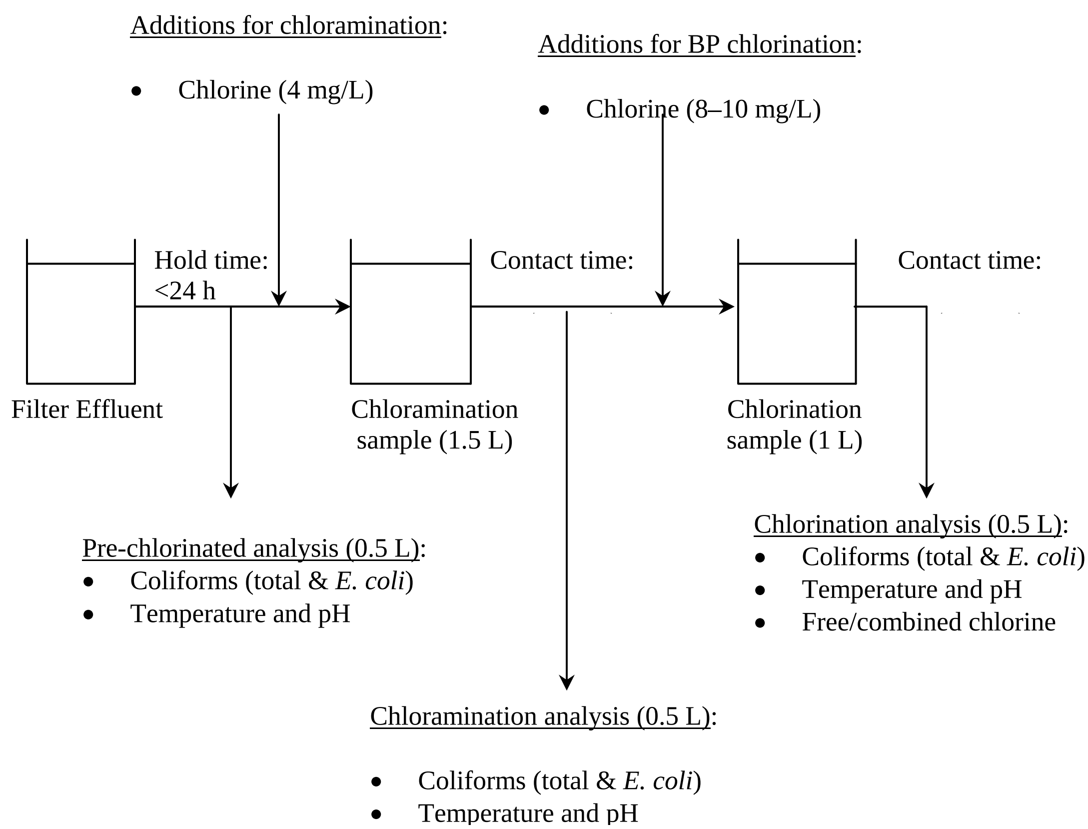
Bench-Scale Disinfection Test

Because the study only involves two individual filters, it is not possible to test disinfection at the full-scale because full-scale disinfection occurs after effluent from all filters (operated at multiple loading rates) are mixed. As a result, a laboratory test of disinfection that mimics the full-scale recycled water disinfection process as closely as possible will be carried out, which involves chloramination followed by breakpoint chlorination. The lab will have 21 samples per filter run (per day) when running chloramination and breakpoint chlorination test.

- Use IDEXX Colilert method with Quanti-tray 2000 for total coliform and *E. coli* analysis
- Amperometric titration method (standard methods 4500-Cl) is used to determine chlorine concentrations.
- *Bench-Scale Disinfection*
  - Goal is to mimic full-scale disinfection as closely as possible
    - *Chemical addition*: ammonia & chlorine (chloramination), followed by more chlorine (breakpoint chlorination). Doses based on full-scale configuration
    - *Contact time and doses*: Based on full-scale design when all filters are operating at 7.5 gpm/ft^2.
  - Bring samples to **room temperature** before adding chlorine.
  - Provide **constant mixing** during the bench scale disinfection test.
  - Measure temperature and pH at chlorine concentration measurement points.
  - When taking coliform samples during bench disinfection, immediately add sodium thiosulfate to dechlorinate.

Bench-Scale Disinfection Procedure Flow Chart

For both batch disinfection samples collected at each filter run, perform the following analysis:



FLEWR Phase II Protocols: MRWPCA

Study Purpose

The purpose of this study is to determine both the impact of higher loading rates on tertiary filter performance and the feasibility of operating the filters at such rates. A pilot-scale study at the MRWPCA water reclamation facility tested loading rates from 2.5 to 12.5 gal/ft^2-min. Operation at filter rates 5 and 7.5 gal/ft^2-min produced equivalent water with average effluent turbidity <2 NTU, when coagulant/polymer doses were optimization specific to loading rate. Based on these pilot results, the CDPH is allowing full-scale loading rate testing, on a plant-by-plant basis, at treatment plants participating in Filter Loading Rate Evaluation for Water Reuse (FLEWR) Phase II study. This Phase II study will compare operation at 5 and 7.5 gal/ft^2-min using actual tertiary filters producing "Title-22" reclaimed water. Various metrics will be used to evaluate filter performance, including:

- Turbidity removal
- Particle removal (2–20 μm)
- Removal of coliform organisms
- Filter effluent disinfectability of coliform bacteria

In addition, feasibility of higher rates will be evaluate using the following operational data:

- Filter run length
- Filter production capacity
- Coagulant doses
- Clean bed head loss and changes backwash requirements

There are two basic filter testing configurations for this study: (1) simultaneous testing, where two filters receive the same influent water but are operated at different loading rates; or (2) consecutive testing, where only one test filter is used and the test loading rate is switched after some number of filter runs. Filter effluent disinfectability is monitored through either the full-scale disinfection system or through laboratory batch testing. The operational protocols are specific to the treatment plant and are dependent on the filter loading rate control scheme, secondary effluent quality, and other operational requirements. The following protocols describe the requirements for filter operations, instrumentation, data recording, laboratory analysis, and data reporting for MRWPCA.

Filter Operation

Operating Configuration

Phase II at MRWPCA will follow the consecutive testing mode, where only one full-scale filter (Filter #6) is designated as the test filter, and only one loading rate is tested at a time. This configuration allows the plant operator to optimize the coagulant dose specific to the test rate (5.0 or 7.5 gal/ft^2-min). The loading rate on the testing filter is held at a constant set point for the entire filter run, until the filter is shutdown (regardless of the tertiary plant flow). All other non-testing filters can operate at variable flow rates (up to a set point of 7.5 gal/ft^2-min) to accommodate the fluctuations in plant flow. To study the effect of higher loading rates on the filter backwashing requirements, one filter (Filter #1) is designated as the control filter, and this filter is never operated at a set point above 5.0 gal/ft^2-min for the entire FLEWR Phase II test period. Table 1 displays the role and requirements for each full-scale filter during the FLEWR Phase II test period.

Table C-1. Phase II Role of MRWPCA Full-Scale Filters

Filter	Phase II Role	Flow rate control	Rates (gal/ft^2-min)	Online instruments	Sampling
1	Backwash control	Variable	5.0 or less	TB	none
2	none	Variable	7.5 or less	TB	none
3	none	Variable	7.5 or less	TB	none
4	none	Variable	7.5 or less	TB	none
5	none	Variable	7.5 or less	TB	none
6	Test Filter	Constant	5.0 or 7.5	TB & PC	Grab Samples

In Run 1, the test filter is operated at a constant set point of 5.0 gal/ft^2-min. After meeting one of the test filter shutdown criteria (see the following), the filter is backwashed following the plant's standard backwashing procedure. The test filter is then restarted (using the test filter start-up listed in the following paragraph) at the other test rate; for example, in Run 2, the test filter should be operated at a constant set point of 7.5 gal/ft^2-min. After meeting the shutdown criteria at this rate, the rate is switched again. The test rate is alternated between every run (thus, all odd numbered runs are set at 5.0 gal/ft^2-min, and even numbered at 7.5 gal/ft^2-min).

Test Filter Start-Up

After backwashing, the test filter is started up with the following conditions:

- *Start time distribution*: Filter runs are started in either the morning (generally between either 8:30 and 10:00AM) or later afternoon (between 3:30 and 6:30 PM) to ensure that adequate data is collected from all times throughout the day at both loading rates.
- *Ramping up to set point rate*: At the plant operator's discretion, the loading rate can be increased gradually during filter start-up to minimize the effect of filter ripening. The testing set point should be reached as soon as possible and within 30 min of filter start-up.
- *Clean bed head loss*: Immediately after reaching the target set point in the test filter, the operator notes the filter head loss value (defined as the CBHL) in the log.

Coagulation Control Strategy

During the FLEWR Phase I pilot-scale testing at MRWPCA, a higher coagulant dose was required for filters operating at higher loading rates to produce an equivalent filter effluent. It is expected that during Phase II, a higher coagulant dose will be required for the higher loading rate to produce an equivalent filter effluent. The following control strategy should be followed:

- *Coagulant dose selected based on test filter performance*: During FLEWR testing, the coagulant dose is selected and optimized based on effluent turbidity of the testing filter (Filter #6). The coagulant dose is increased or decreased to maintain a FILTER RUN AVERAGE effluent turbidity of 1.90 NTU. Note that this is different from the 24h average turbidity.
- *Use minimum coagulant dose*: Use the minimum coagulant dose required to meet the treatment objective in the test filter. Do not increase the coagulant dose if the test filter is exceeding the treatment objective.
- *When to ignore this strategy*: If following this strategy would lead to a violation of any of the permit or waiver conditions (shown in the following) for any of the full-scale filters, then appropriate action should be taken as to not violate any permitting/waiver conditions.

Waiver Conditions

The following waiver conditions are followed for filters operating at >5.0 gal/ft^2-min:

- The 24-h average effluent turbidity from each filter is less than 2.0 NTU.
- The instantaneous filter effluent turbidity does not exceed 2.5 NTU for more than 5% of the time.
- The instantaneous filter effluent turbidity never exceeds 5 NTU.
- Loading rate set point does not exceed 7.5 gal/ft^2-min, and the instantaneous loading rate should not exceed 7.8 gal/ft^2-min.

Test Filter Shutdown Criteria

The filter is shutdown and the backwashed after reaching any of the following:

- *Turbidity exceedance*: If the performance of the test filter is approaching any of the waiver conditions listed earlier, then the filter is shutdown.
- *Terminal head loss*: The filter is shutdown after reaching maximum head loss.
- *24-h maximum run-time*: If the filter run time reaches 24 h, the filter is shutdown.

After the filter flow has stopped, the test filter is not restarted until the filter is backwashed.

Testing Period

- *Preliminary data collection*: A preliminary data set of 10 filter runs will be initially collected. The FLEWR team will analyze this data and provide feedback prior to further testing.
- *Required number of runs*: Based on the preliminary data set, the FLEWR team will estimate the minimum number of testing runs required at each treatment plant to achieve the statistical requirements of the study.

Monitoring for Backwash Changes and Other Operational Changes

- *Monitoring backwash requirements*: Although no changes in backwashing requirements are expected, it is necessary to monitor backwash effectiveness and, if needed, to make changes to the backwashing procedure. A statistically increasing trend in the CBHL is used as an indicator that backwashing changes are needed. Record information on the backwashing procedure and note any procedural changes.
- *Other operational changes*: It is important to avoid any unnecessary process changes during the testing period. If changes to the tertiary system are needed (such as a change in coagulant type), please discuss with the FLEWR team before making process changes. Also, please note and report any abnormalities and/or plant maintenance activities that affected the secondary effluent water quality or tertiary treatment.
- *Suspension of FLEWR testing*: If for any reason Phase II testing must be suspended, stop Phase II testing and resume normal/necessary plant operations. When able, please notify the FLEWR team.

Instrumentation

Required Online Instrumentation

- *Turbiditimeter*: Hach 1720D or 1720E or equivalent
  - Flow rate though the turbidimeter is 750 mL/min
- *Particle Counter*: Hach PCX 2200 or equivalent
  - Set with continuous size ranges >2, >3, >4, >5, >7, >10, >15, >20 μm
  - Instrument flow rate should be 100 mL/min
  - Particle counter dilution system on the secondary effluent sampling line, if total particles (>2 μm) is greater than 16,000 particles/mL

Instrument Locations

The instrumentation is configured to monitor water quality in the following locations:

- *Secondary effluent*: Particle counter (with dilution) and turbidimeter (at MRWPCA in pilot plant)
- *Coagulated/flocculated filter influent*: Particle counter (with dilution) and turbidimeter
- *Test filter effluent*: Particle counter and turbidimeter
- *Combined filter effluent*: Particle counter and turbidimeter

Instrument Calibration and Cleaning

Proper instrument calibration is required. When feasible, ALL CLEANING AND CALIBRATIONS should be completed WHEN THE TEST FILTER IS SHUTDOWN.

- Turbidimeters
  - Calibrated weekly using HACH Formazin standards of 0 and 20 NTU
  - Secondary effluent and coagulated/flocculated turbidimeter cleaned daily
  - Filter effluent turbidimeters is cleaned weekly
- Particle counters

- o Annual calibrations are performed by Hach factory technician
  - o Secondary effluent particle counter is cleaned daily
  - o Filter effluent particle counters are cleaned weekly
  - o For all particle counters, flow rates are checked after each cleaning (flow should be 100 mL/min); adjustments are made if flow is *not* between 98 –102 mL/min
- pH probe
  - o Manufacturer guidelines are followed for cleaning and calibration (likely calibrated weekly—depends on model)

<u>Data Recording</u>

The following data sets are recorded and reported in the specified units:

Continuously Recorded Data

The following data are recorded continuously (recording frequency dependent of plant configurations; at least every 5 min, preferably every 1 min):

- Secondary effluent turbidity (NTU)
- Post coagulation turbidity (NTU)
- Test filter effluent turbidity (NTU)
- Secondary effluent particle counts (>2, 3, 4, 5, 7, 10, 15, and 20 µm) (#/mL)
- Test filter effluent particle counts (>2, 3, 4, 5, 7, 10, 15, and 20 µm) (#/mL)
- Test filter loading rate (gal/ft^2-min)
- Test filter run time (h)
- Test filter cumulative volume treated (gal/ft^2)
- Test filter head loss (ft)
- pH
- Coagulant/polymer dose(s) (mg/L)
- Chlorine dose (mg/L) or UV dose (mW/m^2)
- Chlorine residual (mg/L)

Data Collected Once Per Run

There are also operation data that must be recorded for each filter run. These data are entered into the FLEWR Phase II operator spreadsheet Total run time (h)

- Total volume treated (gal/ft^2)
- Clean bed head loss (ft): head loss at filter start-up following a backwash, once the loading rate has reach the target value
- Head loss at shutdown (ft)
- Shutdown cause (breakthrough, head loss, time, other)
- Backwashing length (min)
- Volume used for backwashing (gal/ft^2)
- Notes on any process changes or disruptions

Laboratory Analysis

Grab Sampling

Grab samples are collected in **three series** during each filter run at multiple sampling locations (Also see “Grab Sampling Flow Chart”), except for the disinfection sample that is only collected once:

- *Sampling locations and volumes*:
  - Secondary effluent (before coagulation and pre-chlorination) (100 mL)
  - Coagulated/flocculated water (100 mL)
  - Test filter effluent (100 mL)
- *Sampling time periods*:
  - Begin of filter run (between 1–2 hrs after filter start-up)
  - Middle of filter run (when test filter head loss is between 4–7 ft or alternatively, somewhere in between first and last sample)
  - End of filter run (when test filter head loss is between 7–9 ft, or alternatively, ~1 hr before filter shutdown)
- *Sample collection*:
  - Flush sample line before sample collection
  - Use a sterile sampling container that already contains sufficient sodium thiosulfate to dechlorinate sample
- *Sample Storage:*
  - Samples are refrigerated after collection
  - Lab analysis begins as soon as possible, but not longer than 24 h after collection
- *Batch Disinfection Sample Collection*
  - Sampled <u>once</u> per filter run (during lab hours), while test filter is in operation
  - Sampled at filter effluent
  - Collect sample in sterile container (without sodium thiosulfate)
  - Record sample collection time and immediately bring sample to lab for immediate bench-scale test (no refrigeration necessary)

Batch Disinfection

The batch disinfection is designed to mimic the full-scale disinfection processes as closely as possible.

- Prepare a concentrated sodium hypochlorite stock solution (~15,000 mg/L) and keep refrigerated when not in use.
- Use IDEXX Colilert[21] method with Quantitray 2000 for total coliform and *E. coli* analysis.
- Use the iodometric titration (Standard methods 4500-Cl C) to determine combined chlorine concentration.
- When taking coliform samples during bench disinfection, use sterile 100-mL Colilert bottles containing sodium thiosulfate (sufficient to dechlorinate sample).
- Perform batch disinfection in a clean and sterile reactor beaker/jar.
- Provide **constant low-speed mixing** during the bench-scale disinfection test using a clean and sterile stir bar.

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[21] Method may change if a significant number false positives are observed.

Bench-Scale Disinfection Procedure (also see Batch Disinfection Flow Chart)

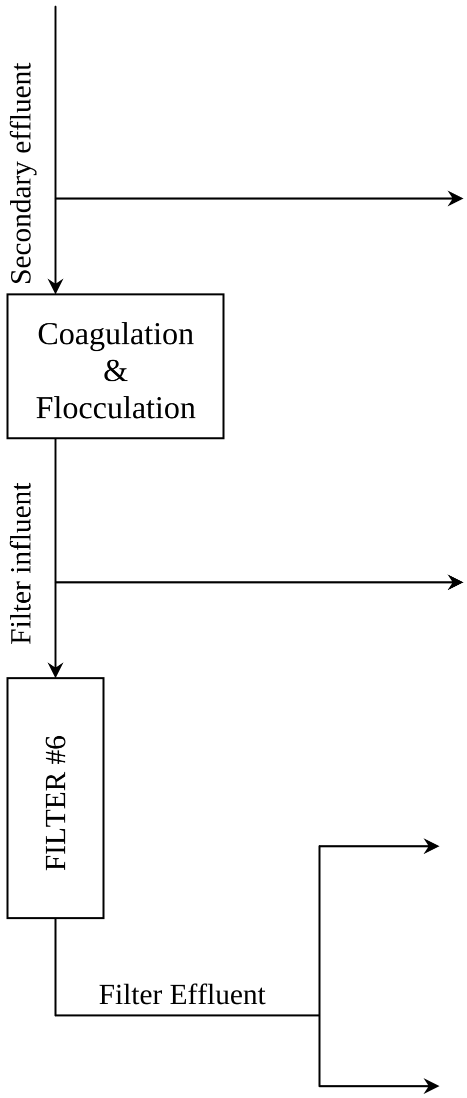
1. *Determine initial chlorine dose*: Use the chlorine dose currently applied at the full-scale system (call operations before each test to check dose). This full-scale dose is typically 11.5 or 12 mg/L. Our goal is to have a 10-mg/L chlorine residual in the batch reactor after 120 min. Calculate the volume of sodium hypochlorite stock to add to batch reactor:

$$\frac{\text{Desired Chlorine Dose (mg/L)}}{\text{Stock Concentration (mg/L)}} * \text{Volume reactor (mL)} = \text{Volume of stock to add (mL)}$$

2. *Obtain sample*: Collect/receive the tertiary filter effluent sample; divide the 1-L sample into two 500-mL samples (one for batch disinfection; one to take the initial filter effluent sample measurements).
3. *Chlorine addition*: To the first 500-mL, add chlorine and provide an initial rapid mixing by vigorously shaking of the chlorine reactor in a sealed container for 30 seconds. After initial mixing, continuously stir reactor at a low speed using a magnetic stir bar to keep the reactor well mixed. Continue this low-speed mixing for the remainder of the experiment. Record chlorine addition time.
4. *Analysis on filter effluent*: To the second 500-mL sample (no chlorine added), measure the initial pH, temperature, and chlorine residual.
5. *Final sample and measurements:* At 120 min, collect 100 mL coliform sample from the reactor in sampling bottle with sodium thiosulfate. Then measure final pH, temperature, and chlorine residual of water remaining in the reactor. Record the sample collection time.
6. *Process coliform sample*: Use the IDEXX Colilert with the Quantitray-2000.

MRWPCA FLEWR Grab Sampling Flow Chart

- For each filter run, there will be **3 sampling periods** spread over the run time.
- For **each** sampling period, the following samples should be collected:



Secondary effluent sample

- Collected 3 times per run
- 100-mL sample collected (with sodium thiosulfate)
- Colilert analysis for total coliforms and *E. coli* (dilution required)
- Store in lab refrigerator until analysis

Coagulated/Flocculated filter influent sample

- Collected 3 times per run
- 100-mL sample collected (with sodium thiosulfate)
- Colilert analysis for total coliforms and *E. coli* (dilution required)

Filter effluent sample

- Collected 3 times per run
- 100-mL sample collected (with sodium thiosulfate)
- Colilert analysis for total coliforms and E. coli (dilution required)

Batch Disinfection Sample

- Collected 1 time per run (during lab hours)
- 1-L sample collected (not dechlorinated)
- Immediate bench-scale disinfection experiment

Total number of Colilert samples per run

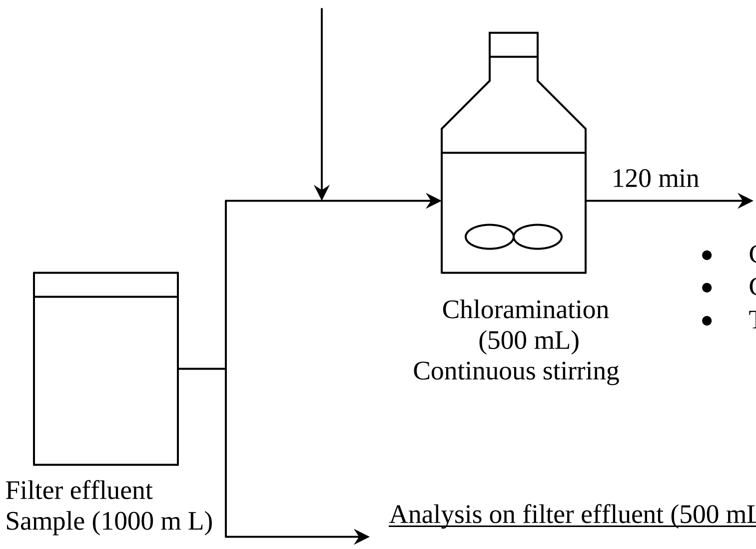
Secondary effluent x 3 sample times/run	=	3 samples/run
Filter influent x 3 sample times/run	=	3 samples/run
Filter effluent x 3 sample times/run	=	3 samples/run
Batch disinfection x 1 sample/run	=	1 sample/run
Total number of samples	=	10 samples/run

MRWPCA FLEWR Batch Disinfection Flow Chart

For the batch disinfection sample collected **each filter run**, perform the following analysis:

Chlorine addition:

- Determine chlorine dose from operations
    - (typically 11.5 or 12 mg/L)
- Add stock sodium hypochlorite (mL)
- Provide initial vigorous mixing
- Record start time



At 120 min:

- Coliforms (total & *E. coli*)
- Combined chlorine residual
- Temp and pH

Analysis on filter effluent (500 mL)

- Combined chlorine residual
- Temp and pH

FLEWR Phase II Protocol: Virus Disinfectability

Background

The Filter Loading Evaluation for Water Reuse (FLEWR) study was designed to determine the impact of loading rate on tertiary filter performance at full-scale water reclamation facilities. A pilot-scale study at the MRWPCA water reclamation facility tested loading rates from 2.5–12.5 gal/ft^2-min. Operation at filter rates 5 and 7.5 gal/ft^2-min produced equivalent water with average effluent turbidity <2 NTU, when coagulant/polymer doses were optimized for each loading rate (Williams et al. 2007[22]). During the pilot-scale studies, the ability to disinfect the filter effluent was not affected by loading rate. Chloramine disinfection with a 450 mg-min/L C*T resulted in approximately 0.6 log reduction in MS2.

During Phase II of the FLEWR project, five (or more) full-scale tertiary filters are being operated at loading rates of 5 and 7.5 gal/ft^2-min. The equivalence of the effluent quality at the two loading rates will be evaluated according to the following criteria:

- No significant increase (**) in mean turbidity of filter effluent
- No significant increase (**) in mean concentration of 2–5 and 5–15 µm particles in filter effluent
- No significant decrease in ability to disinfect filter effluent

** Significant increase is defined by the percentage = $\dfrac{0.2\text{ NTU}}{\text{NTU produced at 5.0 gal/ft}^2\text{-min}}$

To assess the ability to disinfect filter effluent, the inactivation of total coliform bacteria and *E.coli* by chlorine or UV is measured in filter effluents. At the request of CDPH, following a meeting on October 10, 2007, the disinfection experiments have been expanded to include MS2 coliphage, a common surrogate for human enteric viruses. The experimental design and protocol for these experiments is outlined in the following.

Experimental Design

The objective of these experiments is to determine if increasing the tertiary filter loading rate of full-scale filters from 5 to 7.5 gal/ft^2-min will impact subsequent disinfection of viruses. The objective will be achieved through a series of batch disinfection experiments. Samples will be collected from each full-scale tertiary filtration system participating in the FLEWR study. The samples will be spiked with MS2 coliphage to a concentration of approximately 10^7 PFU/mL. Samples will be well mixed and divided into reactors that will receive different chlorine doses such that there is a 90-min residual chlorine concentration of 5, 13.3, and 27 mg-Cl_2/L, corresponding to C*T values of 450, 1200, and 2400 mg/L-min. In addition, for

[22] Williams, G. J.; B. Sheikh, R. B. Holden; T. J. Kouretas; K. L. Nelson (2007) "The impact of increased loading rate on granular media, rapid depth filtration of wastewater" *Water Research*, 41(19), 4535–4545.

treatment plants that do not nitrify, one additional chlorine dose will be selected to achieve breakpoint chlorination. All doses will be tested in triplicate and statistical analysis will be performed to test for differences in the inactivation curves of CT versus log inactivation. This analysis will be conducted at U.C. Berkeley on one filter effluent sample from each loading rate at each plant.

Protocol

1. ***Sample collection***: Plant staff collects **one 1-L** filter effluent sample from test filter(s) operating at test rates 5 and 7.5 gpm/ft^2. Samples are collected in sterile 1-L plastic bottles. Samples are refrigerated immediately after collection, until shipping.
2. ***Sample transport***: Samples are packed in cooler with sufficient ice packs to keep cool and are shipped on the same day as collection, via overnight courier. Analysis will begin within 24 h upon collection. Alternatively, samples can be frozen at time of collection, shipped frozen, and thawed before analysis (in cases where analysis cannot be preformed within 24 h).
3. ***Reagent and glassware preparation***:
    a. MS2 working solution is prepared by diluting an MS2 stock to 10^{10} PFU/mL in phosphate buffer solution with an ionic strength of 10 mM.
    b. *E. coli* stock solution is grown in tryptone broth (with ampicillin and streptomycin) inoculated with antibiotic resistant *E. coli* 16–24 h before plating the samples.
    c. Sodium Hypochlorite (NaOCl) stock solution is diluted in deionized water to a concentration of ~2 mg-Cl_2/mL and stored at 4ºC.
    d. Sodium thiosulfate solution (STS) is prepared by dissolving sodium thiosulfate crystals in DI water to a concentration of 16 mg/mL and sterilizing by autoclave.
    e. All reagents for the chlorine residual test are prepared following the Standard Methods 4500-Cl C (Clesceri et al. 1998), except that there is a 10-fold dilution of the iodine (prepared the day of the experiment) to increase method resolution. The other reagents are phenylarsine oxide solution, potassium iodide, starch indicator, and an acetate buffer solution.
    f. All glassware for the experiment is washed with soap and thoroughly rinsed with DI water and sterilized in the autoclave. Glassware for the chlorine residual measurement is not sterilized.
4. ***NH_3^+ concentration***: Determine ammonia concentration for each sample using HACH kit following the HACH protocol.
5. ***Measure chlorine stock concentration***: Following standard methods 4500-Cl C, measure the chlorine concentration of the stock solution.
6. ***Bring samples to room temperature***: After bringing samples to room temperature, record the pH and temperature for each sample.
7. ***Measure chlorine demand in samples***: In cases where full-scale filters are pre-chlorinated (MRWPCA and LACSD), measure the chlorine residual and assume the chlorine demand has been consumed. For unchlorinated samples, dose a 50 mL sample to 10 mg-Cl_2/L chlorine, wait 90-min, and measure the chlorine residual to determine the chlorine demand.
8. ***Collect the "native phage" sample***: Collect 10 mL of the each sample for quantification of the indigenous F+ male-specific phage.
9. ***Seed MS2 into samples***: For each filter effluent sample, pour 750 mL of sample into a glass media jar. Pipette in 1.5 mL of the *MS2 working solution (~10^{10} PFU/mL)*,

creating a final concentration of ~2*10^7 PFU/mL. Cap the media jar and mix sample well by vigorously shaking the bottle for 30 s.

10. ***Collect "initial MS2" sample***: Collect three 10-mL of each sample for quantification of the seeded concentration of MS2. If the filter samples were pre-chlorinated, add sodium thiosulfate to the 10-mL samples to dechlorinate. Determine the necessary volume of STS using the following equation:

$$\left(\text{Vol. of STS } mL\right) = 13.5 * \frac{\left(\text{Cl residual } mg/L\right)}{\left(\text{Conc Na}_2\text{S}_2\text{O}_3 \text{ in STS } mg/L\right)} * 10mL$$

11. ***Divide samples***: Divide each sample into nine 50-mL aliquots (12 aliquots if doing break point chlorination). Use 75-mL glass test tubes as the disinfection reactors.
12. ***Dose aliquots with chlorine:***
    a. Each of the three target residuals (5, 13.3, 27 mg/L) and the breakpoint dose (if applicable) is tested in triplicate, requiring up to 12 aliquots for each sample.
    b. If applicable, use the following equation to calculated the breakpoint chlorination dose:

$$\left(\text{Breakpoint dose } mg\text{-}Cl_2/L\right) = 9 * \left(\text{Conc. of NH}_3 \ mg\text{-}N/L\right)$$

    c. Use the following equation to calculated the volume of NaOCl stock solution to add to the 50-mL aliquots:

$$\left(\text{Vol. } of \text{ NaOCl sol. } mL\right) = \frac{\left(\text{Cl}_2 \text{ dose } mg/L\right) + \left(\text{Cl}_2 \text{ demand } mg/L\right)}{\left(\text{Conc. } of \text{ NaOCl sol. } mg/L\right)} * 50mL$$

    d. Note the time and add the specified chlorine dose to each aliquot. Immediately after each chlorine addition, cap test tube (or cover with parafilm) and vigorously mix for 10s by vortexing at maximum speed.
    e. After samples are dosed and mixed, place on a shaker table to keep well mixed during contact time.
13. ***Wait 90 min contact time***.
14. ***Pre-add sodium thiosulfate to sample containers while waiting.*** Use the equation in step 10 to determine the appropriate volume of STS to dechlorinate the 10-mL disinfection samples after 90 min. Pipette the appropriate STS volume for each sample into labeled sample containers.
15. ***Collect samples at 90 min***: After 90 min, pipette 10 mL of each reactor into the appropriate sample container. Immediately after sample collection, vortex sample container for 3 s to mix STS with sample.
16. ***Measure pH and temperature of the sample remaining in the reactor***. Only minimal variation in pH and temperature will occur between replicates, so only one measurement is required per chlorine dose per sample.
17. ***Measure chlorine residual of the sample remaining in the reactor***. Use *Standard Methods 4500-Cl C Iodometric Method II* to quantify the chlorine residual in all samples.
18. ***Plate MS2 samples***: Perform necessary dilutions and plate appropriate concentrations using the double layer agar methods (Adams, 1959) to enumerate the MS2 dilutions in all samples and in the *MS2 working solution*. Control plates are required to ensure

there is no MS2 contamination in solutions of sodium thiosulfate, PBS, and *E. coli*. After all samples are plated, incubated at 37ºC.

19. ***Read Plates***: Read plates after 12–24 h of incubation and record the number of plaques that formed on each plate.

APPENDIX D. DATA SUMMARY FROM FULL-SCALE TESTING

Table D1: Filter Runs at 5.0 gal/ft^2-min (Monterey Condition A)

Run #	Filter Start Time	Filter Stop Time	Run Time (h)	Loading Rate (gpm/ft^2)	Terminal HL (ft)	Coagulant Dose (mg/L)	SE[a] Turbidity (NTU)	CG[a] Turbidity (NTU)	FE[a] Turbidity (NTU)	SE 2-5 mm particles per mL	SE 5-15 mm particles per mL	CG 2-5 mm particles per mL	CG 5-15 mm particles per mL	FE 2-5 mm particles per mL	FE 5-15 mm particles per mL	Volume Treated (gal/ft^2-run)	Est. Daily Production Capacity (gal/ft2-d)
P2	6/27/07 7:02	6/28/07 6:26	23.4	5.01	11.5	3.1	3.01	n/a[b]	1.58	14575	6353	n/a[b]	n/a[b]	1437	212	7031	6243
P3	6/28/07 8:21	6/29/07 8:20	24.0	5.01	8.6	1.9	3.29	n/a[b]	1.54	12525	5829	n/a[b]	n/a[b]	1626	222	7213	6271
P5	7/9/07 17:30	7/10/07 17:30	24.0	5.01	7.2	3.7	3.84	n/a[b]	1.58	14139	7098	n/a[b]	n/a[b]	1138	194	7216	6270
P8	7/16/07 16:42	7/17/07 16:47	24.1	5.00	8.6	3.6	3.66	n/a[b]	1.61	12042	5791	n/a[b]	n/a[b]	1441	245	7222	6259
P10	7/19/07 8:31	7/20/07 8:36	24.1	5.00	8.1	3.2	3.42	n/a[b]	1.69	14019	5709	n/a[b]	n/a[b]	1379	235	7231	6267
Average	--	--	23.9	5.0	8.8	3.1	3.44	n/a[b]	1.60	13460	6156	n/a[b]	n/a[b]	1404	222	7183	6262
+/- 95% CI	--	--	0.4	0.0	2.0	0.9	0.40	n/a[b]	0.07	1375	726	n/a[b]	n/a[b]	218	25	106	15

Table D2: Filter Runs at 7.5 gal/ft^2-min (Monterey Condition A)

Run #	Filter Start Time	Filter Stop Time	Run Time (h)	Loading Rate (gpm/ft^2)	Terminal HL (ft)	Coagulant Dose (mg/L)	SE Turbidity (NTU)	CG Turbidity (NTU)	FE Turbidity (NTU)	SE 2-5 mm particles per mL	SE 5-15 mm particles per mL	CG 2-5 mm particles per mL	CG 5-15 mm particles per mL	FE 2-5 mm particles per mL	FE 5-15 mm particles per mL	Volume Treated (gal/ft^2-run)	Est. Daily Production Capacity (gal/ft2-d)
P1	6/26/07 8:52	6/26/07 23:24	14.6	7.58	11.5	1.7	2.91	n/a[b]	1.55	13195	5610	n/a[b]	n/a[b]	1541	300	6619	9132
P4	6/29/07 9:47	6/30/07 3:07	17.4	7.60	11.4	1.7	2.97	n/a[b]	1.53	10255	4871	n/a[b]	n/a[b]	2005	346	7911	9424
P6	7/11/07 17:59	7/12/07 11:42	17.7	7.48	11.8	4.0	3.95	n/a[b]	1.73	12430	7494	n/a[b]	n/a[b]	1408	369	7945	9292
P7	7/12/07 16:58	7/13/07 10:36	17.6	7.50	11.4	3.4	3.33	n/a[b]	1.69	12819	6240	n/a[b]	n/a[b]	1391	357	7938	9317
P9	7/18/07 6:13	7/18/07 21:30	15.3	7.50	11.5	3.0	3.08	n/a[b]	1.71	11205	4848	n/a[b]	n/a[b]	1704	322	6879	9105
Avg	--	--	16.5	7.5	11.5	2.7	3.25	n/a[b]	1.64	11981	5813	n/a[b]	n/a[b]	1610	339	7458	9254
+/- 95% CI	--	--	1.8	0.1	0.2	1.3	0.53	n/a[b]	0.12	1515	1370	n/a[b]	n/a[b]	316	34	812	166

Table D3: Statistical comparison of filter preformance between 5 and 7.5 gal/ft^2-min (Monterey Condition A)

Comparative Statistics	Run Time (h)	Loading Rate (gpm/ft^2)	Terminal HL (ft)	Coagulant Dose (mg/L)	SE TB (NTU)	CG TB (NTU)	FE TB (NTU)	SE 2-5 mm particles per mL	SE 5-15 mm particles per mL	CG 2-5 mm particles per mL	CG 5-15 mm particles per mL	FE 2-5 mm particles per mL	FE 5-15 mm particles per mL	Volume Treated (gal/ft^2-run)	Est. Daily Production Capacity (gal/ft2-d)
Average difference between 5 and 7.5 gpm/ft^2	-7.4	2.5	2.7	-0.4	-0.20	--	0.04	-1479	-343	--	--	205	117	276	2992
Percent Change from 5 to 7.5 gpm/ft^2	-31%	50%	31%	-12%	-6%	--	3%	-11%	-6%	--	--	15%	53%	4%	48%
Is the 7.5 gpm/ft^2 rate statistically worse?[c]	yes	--	--	no	no	--	no	no	no	--	--	no	yes	no	no
Total Sample Size (Number of Runs)	10	10	10	10	10	--	10	10	10	--	--	10	10	10	10

[a] SE = Secondary Effluent; CG = Coagulated/Flocculated Filter Influent; FE = Filter Effluent

[b] Prior to installation of coagulated water monitoring equipment

[c] Significance level defined as a = 0.05 for two sample hypothesis test (H_0: $m_5 \geq m_{7.5}$; H_A: $m_5 < m_{7.5}$) using *t* test

Table D4: Filter Runs at 5.0 gal/ft²-min (Monterey Condition B)

Run #	Filter Start Time	Filter Stop Time	Run Time (h)	Loading Rate (gpm/ft²)	Terminal HL (ft)	Coagulant Dose (mg/L)	SE[a] Turbidity (NTU)	CG[a] Turbidity (NTU)	FE[a] Turbidity (NTU)	SE 2-5 µm particles per mL	SE 5-15 µm particles per mL	CG 2-5 µm particles per mL	CG 5-15 µm particles per mL	FE 2-5 mm particles per mL	FE 5-15 mm particles per mL	Volume Treated (gal/ft²-run)	Est. Daily Production Capacity (gal/ft2-d)
1	9/6/07 8:43	9/7/07 8:36	23.9	5.01	10.9	7.0	4.66	n/a[b]	1.83	15182	6793	n/a[b]	n/a[b]	1137	279	7175	6263
2	9/11/07 8:04	9/12/07 8:02	24.0	4.98	9.6	4.1	3.93	6.44	1.48	13366	4788	8361	4768	1101	227	7155	6224
3	9/17/07 16:59	9/18/07 14:48	21.8	5.01	11.4	6.9	5.07	8.17	1.70	20459	6829	14444	5295	1072	282	6547	6175
5	9/19/07 17:15	9/20/07 16:03	22.8	5.01	11.6	5.6	4.64	7.59	1.76	19698	6916	13549	5213	1210	299	6851	6221
9	9/26/07 9:06	9/27/07 9:35	24.5	5.01	10.1	5.7	3.09	7.27	1.81	16250	4760	8546	3926	892	209	7352	6287
10	9/27/07 10:53	9/28/07 10:26	23.6	5.01	11.4	6.1	3.00	7.77	1.85	14712	4141	8721	3902	789	167	7072	6248
11	10/1/07 16:14	10/2/07 16:14	24.0	5.01	13.4	8.8	4.38	8.30	1.83	15228	5042	12748	3925	970	215	7210	6267
12	10/3/07 16:36	10/4/07 16:36	24.0	5.01	11.2	7.1	4.13	7.75	1.73	13648	5197	8686	4102	662	145	7212	6270
18	4/22/08 8:15	4/23/08 7:51	23.6	5.00	7.3	3.5	4.03	6.45	1.86	14771	8151	6986	6299	1254	231	7075	6237
20	4/29/08 8:14	4/30/08 7:36	23.4	5.01	11.5	5.7	4.37	7.63	1.93	16004	7869	10922	8517	1147	308	7018	6241
22	5/1/08 8:54	5/2/08 9:30	24.6	5.00	10.5	4.7	4.15	7.00	1.81	12143	5264	7777	7381	1132	260	7377	6277
24	5/8/08 8:16	5/9/08 5:40	21.4	5.00	11.6	5.7	4.91	6.14	1.80	13148	5968	5574	5271	961	206	6424	6154
26	5/20/08 18:07	5/21/08 15:57	21.8	4.99	11.6	5.5	4.79	7.63	1.86	12762	4596	8868	6756	1366	348	6528	6150
28	5/27/08 17:46	5/28/08 14:11	20.4	5.00	11.5	7.5	4.53	8.24	1.81	13995	4386	7762	5123	722	159	6125	6101
30	5/29/08 16:32	5/30/08 12:26	19.9	5.00	11.5	6.4	4.63	8.79	1.98	n/a[c]	n/a[c]	7543	4152	1049	241	5969	6073
32	6/10/08 16:17	6/11/08 16:25	24.1	5.00	10.5	4.5	3.45	6.38	1.81	12799	5571	4917	3604	1148	259	7242	6265
34	6/12/08 7:46	6/13/08 5:57	22.2	5.00	11.5	4.5	3.50	7.00	1.87	15152	4356	6309	4935	1157	229	6651	6181
36	6/17/08 8:38	6/18/08 8:47	24.2	5.00	7.9	2.5	3.78	5.77	1.76	14301	5117	13782	4951	1570	281	7251	6269
38	6/24/08 8:44	6/25/08 5:53	21.2	5.01	7.8	2.9	4.35	7.01	1.81	n/a[c]	n/a[c]	6932	6302	1449	270	6354	6147
40	7/8/08 8:44	7/9/08 5:42	21.0	5.01	11.4	6.8	4.37	7.51	1.84	14187	5910	9061	4629	748	194	6299	6138
42	7/10/08 15:26	7/11/08 13:42	22.3	4.99	11.6	6.1	4.19	7.72	1.81	13253	4487	8394	4209	743	182	6665	6173
44	7/16/08 15:58	7/17/08 16:10	24.2	5.00	9.3	4.5	4.19	7.46	1.86	11303	4284	8870	4677	1168	300	7263	6268
46	7/22/08 16:08	7/23/08 16:19	24.2	5.00	9.8	5.2	4.28	7.03	1.84	18526	5739	15347	6738	1545	329	7260	6269
48	7/24/08 15:27	7/25/08 14:02	22.6	5.01	11.5	5.5	4.20	7.36	1.91	11389	4138	7530	5266	860	206	6784	6210
50	7/30/08 8:23	7/31/08 3:37	19.2	5.01	7.7	4.5	3.80	6.82	1.85	12638	4742	4843	4527	1207	161	5778	6048
52	8/5/08 8:23	8/6/08 1:20	17.0	5.00	11.5	4.2	4.29	6.92	1.80	15976	5094	8440	4406	1981	203	5088	5895
54	8/7/08 8:59	8/8/08 3:25	18.4	5.00	11.7	5.8	4.21	7.68	1.82	12394	3701	8617	3851	854	153	5530	5990
56	8/13/08 8:31	8/14/08 8:37	24.1	5.00	9.8	1.4	3.29	4.64	1.52	8380	2804	4093	3928	2159	299	7237	6268
58	8/18/08 16:07	8/19/08 16:17	24.2	4.98	9.3	3.0	3.68	5.75	1.53	7657	2438	7074	4299	1447	234	7223	6238
60	8/20/08 16:07	8/21/08 16:17	24.2	5.00	9.7	2.9	2.94	4.87	1.47	12391	2767	14372	4334	1653	293	7253	6267
62	8/27/08 15:57	8/28/08 16:08	24.2	5.01	6.6	2.6	2.78	4.77	1.56	6931	2903	7718	3512	1224	240	7266	6275
Average	--	--	22.6	5.0	10.4	5.1	4.05	7.00	1.78	13746	4991	8893	4960	1173	239	6782	6196
+/- 95% CI	--	--	0.7	0.0	0.6	0.6	0.22	0.39	0.05	1164	543	1117	449	129	20	218	35

[a] SE = Secondary Effluent; CG = Coagulated/Flocculated Filter Influent; FE = Filter Effluent

[b] Prior to installation of coagulated water monitoring equipment

[c] Data logging equipment failure; secondary effluent water quality not recorded

Table D5: Filter Runs at 7.5 gal/ft²-min (Monterey Condtion B)

Run #	Filter Start Time	Filter Stop Time	Run Time (h)	Loading Rate (gpm/ft²)	Terminal HL (ft)	Coagulant Dose (mg/L)	SE[a] Turbidity (NTU)	CG[a] Turbidity (NTU)	FE[a] Turbidity (NTU)	SE 2-5 µm particles per mL	SE 5-15 µm particles per mL	CG 2-5 µm particles per mL	CG 5-15 µm particles per mL	FE 2-5 mm particles per mL	FE 5-15 mm particles per mL	Volume Treated (gal/ft²-run)	Est. Daily Production Capacity (gal/ft2-d)
4	9/18/07 17:03	9/19/07 4:36	11.6	7.50	11.4	6.2	5.13	8.08	1.38	15311	5071	11470.81	5946.29	1195	323	5196	8602
6	9/20/07 19:03	9/21/07 6:28	11.4	7.47	11.6	7.8	4.55	9.02	1.84	23946	7736	14681	4934	1302	367	5117	8542
7	9/24/07 8:46	9/24/07 21:27	12.7	7.50	9.1	6.9	3.36	6.93	1.47	12917	4814	14773	3745	972	205	5709	8787
8	9/25/07 9:01	9/25/07 22:13	13.2	7.49	11.4	8.8	3.37	7.63	1.43	14408	5101	15256	4006	884	208	5934	8847
13	10/4/07 20:16	10/5/07 8:05	11.8	7.49	11.5	10.9	4.68	9.39	1.41	14230	4208	19987	4474	1375	310	5313	8641
14	10/10/07 8:39	10/10/07 22:22	13.7	7.50	11.3	9.0	4.19	8.08	1.48	13882	5852	14803	3896	1052	245	6174	8928
15	10/11/07 9:12	10/11/07 22:17	13.1	7.49	10.5	8.8	3.86	7.65	1.33	12420	5256	12102	3868	984	227	5882	8832
16	10/16/07 16:00	10/17/07 7:46	15.8	7.48	11.5	8.6	4.14	7.99	1.55	13014	5855	16581	4205	1502	382	7080	9131
17	10/18/07 17:01	10/19/07 5:51	12.8	7.50	11.4	11.6	5.42	8.57	1.55	14918	6316	20674	6221	1120	299	5776	8808
19	4/23/08 9:32	4/23/08 21:31	12.0	7.49	11.4	9.7	3.34	6.54	1.08	12810	5837	10227	5004	836	218	5386	8665
21	4/30/08 9:05	4/30/08 21:30	12.4	7.47	12.1	9.3	3.40	6.83	1.08	14026	5629	7455	5310	763	194	5567	8708
23	5/7/08 9:07	5/7/08 18:23	9.3	7.48	11.8	7.5	3.95	6.98	1.24	13058	6318	6578	4810	820	239	4153	8100
25	5/19/08 8:19	5/19/08 21:41	13.4	7.42	11.6	7.7	3.57	6.49	1.10	18086	4295	9452	4553	973	221	5947	8767
27	5/21/08 19:29	5/22/08 6:32	11.1	7.49	11.7	8.2	5.12	8.63	1.36	20798	7531	10780	7853	957	346	4965	8500
29	5/28/08 18:57	5/29/08 5:55	11.0	7.49	12.3	8.5	4.20	8.22	1.43	n/a[d]	n/a[d]	8785	4824	658	225	4928	8486
31	6/9/08 18:51	6/10/08 4:17	9.4	7.41	12.2	12.5	5.29	9.26	1.38	14083	7281	14097	5559	1132	311	4195	8053
33	6/11/08 19:12	6/12/08 5:19	10.1	7.48	12.5	8.1	4.61	8.48	1.31	13757	5672	7988	4820	729	224	4540	8299
35	6/16/08 8:59	6/16/08 18:17	9.3	7.48	11.4	5.5	2.95	6.16	1.26	11765	3799	5107	2993	555	143	4175	8111
37	6/18/08 9:48	6/18/08 23:19	13.5	7.50	11.5	5.7	2.89	6.21	1.39	10128	3432	6765	3330	704	174	6082	8898
39	6/25/08 10:31	6/25/08 23:29	13.0	7.50	11.5	7.1	4.00	7.32	1.44	n/a[d]	n/a[d]	10034	5090	859	260	5833	8821
41	7/9/08 9:30	7/9/08 22:04	12.6	7.50	11.4	7.5	3.09	6.15	1.35	12002	3756	9599	3418	661	176	5652	8763
43	7/15/08 18:56	7/16/08 7:42	12.8	7.45	11.9	9.7	4.85	8.50	1.54	15451	5734	13637	5007	1160	377	5709	8734
45	7/17/08 17:49	7/18/08 5:02	11.2	7.47	11.6	9.1	4.68	8.40	1.28	20062	6862	16726	5709	833	260	5033	8506
47	7/23/08 18:57	7/24/08 5:07	10.2	7.49	11.6	8.4	4.86	8.04	1.17	15664	5685	13781	5940	539	169	4570	8326
49	7/29/08 19:58	7/30/08 6:55	11.0	7.47	11.6	7.8	5.26	8.71	1.34	18569	8412	9065	6334	582	183	4906	8453
51	7/31/08 9:35	7/31/08 21:47	12.2	7.49	11.5	6.2	3.24	6.36	1.29	11899	4061	7046	3630	552	111	5483	8700
53	8/6/08 10:09	8/6/08 18:39	8.5	7.49	11.3	5.2	2.92	5.88	1.37	12297	3750	6804	4139	1002	176	3821	7903
55	8/12/08 8:57	8/12/08 21:30	12.6	7.50	11.6	4.1	3.42	5.65	1.50	11210	3650	5958	4041	1135	190	5649	8768
57	8/14/08 10:17	8/15/08 0:48	14.5	7.50	11.4	3.1	2.89	5.19	1.47	9223	2951	4440	3156	1056	149	6533	9021
59	8/19/08 17:37	8/20/08 7:15	13.6	7.50	11.7	3.7	3.89	6.42	1.63	13724	3712	7780	4819	1381	263	6137	8918
61	8/21/08 17:33	8/22/08 7:18	13.8	7.48	11.8	4.9	3.00	6.11	1.46	13411	3040	8498	4029	1241	237	6172	8904
Avg	--	--	12.0	7.5	11.5	7.7	4.00	7.41	1.38	14382	5228	10998	4699	952	239	5407	8630
+/- 95% CI	--	--	0.6	0.0	0.2	0.8	0.30	0.43	0.06	1229	561	1572	395	97	26	277	110

Table D6: Statistical comparison of filter preformance between 5 and 7.5 gal/ft²-min (Monterey Condition B)

Comparative Statistics	Run Time (h)	Loading Rate (gpm/ft²)	Terminal HL (ft)	Coagulant Dose (mg/L)	SE TB (NTU)	CG TB (NTU)	FE TB (NTU)	SE 2-5 µm particles per mL	SE 5-15 µm particles per mL	CG 2-5 µm particles per mL	CG 5-15 µm particles per mL	FE 2-5 µm particles per mL	FE 5-15 µm particles per mL	Volume Treated (gal/ft²-run)	Est. Daily Production Capacity (gal/ft2-d)
Average difference between 5 and 7.5 gpm/ft²	-10.6	2.5	1.1	2.6	-0.05	0.42	-0.39	635	237	2105	-261	-221	0	-1375	2433
Percent Change from 5 to 7.5 gpm/ft²	-47%	50%	11%	51%	-1%	6%	-22%	5%	5%	24%	-5%	-19%	0%	-20%	39%
Is the 7.5 gpm/ft² rate statistically worse?[e]	yes	--	--	yes	no	no	no	no	no	yes	no	no	no	yes	no
Does this meet CDPH Equivalency Criteria?[f]	--	--	--	--	--	--	YES	--	--	--	--	YES	YES	--	--
Significance detection ability (%)[e,f]	3%	0%	5%	16%	8%	7%	3%	10%	13%	18%	10%	11.3%	11.3%	4%	2%
Total Sample Size (Number of Runs)	62	62	62	62	62	61	62	58	58	61	61	62	62	62	62

[d] Data logging equipment failure; secondary effluent water quality not recorded

[e] Significance level defined as a = 0.05 for two sample hypothesis test (H_0: $m_5 \geq m_{7.5}$; H_A: $m_5 < m_{7.5}$) using t test

Table D7: Filter Runs at 5.0 gal/ft^2-min (San Jose)

Run #	Filter	Filter Start Time	Filter Stop Time	Run Time (h)	Loading Rate (gpm/ft^2)	Terminal HL (ft)	Turbidity (NTU)		2-5 µm particles per mL		5-15 µm particles per mL		Volume Treated (gal/ft^2-run)	Production Capacity (gal/ft^2-d)
							Influent	Effluent	Influent	Effluent	Influent	Effluent		
P1	B3	3/7/07 11:30	3/8/07 17:44	30.2	5.04	10.0	2.06	0.66	4498	3487	1508	531	9135	7074
P3	B4	3/8/07 6:31	3/9/07 9:25	26.9	5.04	11.5	2.26	0.27	4503	3470	1543	627	8126	7055
P5	B3	3/9/07 8:03	3/10/07 14:49	30.8	5.04	9.9	1.79	0.65	5949	3162	1911	492	9291	7077
P7	B4	3/10/07 3:02	3/11/07 6:50	27.8	5.05	11.5	1.79	0.26	5847	2964	1959	550	8420	7072
P9	B3	3/11/07 6:15	3/12/07 14:47	32.5	5.03	9.9	1.74	0.61	5610	3045	1926	498	9819	7082
P11	B4	3/12/07 2:26	3/13/07 6:29	28.1	5.04	10.5	1.72	0.26	5580	3051	1966	579	8484	7064
P14	B3	3/13/07 8:30	3/14/07 17:31	33.0	5.04	10.0	1.50	0.64	5561	3230	1859	525	9987	7097
P15	B4	3/14/07 1:19	3/15/07 5:14	27.9	5.04	10.1	1.68	0.28	5554	2734	1924	577	8440	7064
P18	B3	3/15/07 10:34	3/16/07 20:16	33.7	5.04	11.2	1.67	0.68	6212	3642	2180	603	10188	7093
P19	B4	3/16/07 0:03	3/17/07 4:14	28.2	5.04	10.1	1.78	0.31	6192	3526	2268	658	8521	7066
P22	B3	3/17/07 12:36	3/18/07 19:48	31.2	5.04	14.6	2.01	0.69	4919	3685	1716	570	9440	7089
P23	B4	3/17/07 22:09	3/19/07 1:21	27.2	5.04	9.9	1.47	0.30	6500	3604	2317	647	8218	7058
P26	B3	3/19/07 11:00	3/20/07 19:11	32.2	5.04	10.1	1.50	0.69	6526	3413	2313	556	9732	7094
P27	B4	3/19/07 18:11	3/20/07 22:14	28.0	5.04	10.0	1.66	0.30	6338	3237	2279	600	8478	7067
P30	B3	3/21/07 10:58	3/22/07 17:30	30.5	5.04	10.4	1.94	0.67	5563	3345	1915	586	9222	7077
P31	B4	3/21/07 12:30	3/22/07 16:01	27.5	5.04	9.9	1.97	0.30	5547	3188	1912	652	8316	7062
P34	B4	3/23/07 6:44	3/24/07 10:03	27.3	5.03	9.1	1.86	0.26	5309	3129	1829	605	8249	7047
P35	B3	3/23/07 9:40	3/24/07 17:02	31.4	5.04	9.2	1.73	0.65	5385	3234	1796	554	9486	7084
P38	B4	3/25/07 2:48	3/26/07 5:14	26.4	5.04	9.7	1.70	0.26	5182	2986	1761	546	7993	7055
P39	B3	3/25/07 7:43	3/26/07 13:09	29.4	5.04	9.5	1.60	0.66	5163	3045	1790	502	8899	7074
Average		--	--	29.5	5.0	10.4	1.77	0.47	5597	3259	1934	573	8922	7073
+/- 95% CI		--	--	1.1	0.0	0.6	0.09	0.09	274	120	110	23	320	7

Table D8: Filter Runs at 7.5 gal/ft^2-min (San Jose)

Run #	Filter	Filter Start Time	Filter Stop Time	Run Time (h)	Loading Rate (gpm/ft^2)	Terminal HL (ft)	Turbidity (NTU)		2-5 µm particles per mL		5-15 µm particles per mL		Volume Treated (gal/ft^2-run)	Production Capacity (gal/ft^2-d)
							Influent	Effluent	Influent	Effluent	Influent	Effluent		
P2	B4	3/7/07 11:30	3/8/07 4:01	16.5	7.54	10.2	2.08	0.33	4561	3803	1515	702	7476	10437
P4	B3	3/8/07 18:27	3/9/07 6:44	12.3	7.55	10.9	2.53	0.69	4550	3486	1583	573	5562	10300
P6	B4	3/9/07 10:47	3/10/07 2:05	15.3	7.55	10.0	1.76	0.27	6057	3478	1902	643	6927	10407
P8	B3	3/10/07 15:43	3/11/07 5:07	13.4	7.54	11.0	1.88	0.64	5859	3107	1998	520	6063	10337
P10	B4	3/11/07 7:44	3/12/07 1:32	17.8	7.55	10.4	1.61	0.24	5655	3232	1901	635	8052	10470
P12	B3	3/12/07 15:32	3/13/07 7:34	16.0	7.53	11.1	1.60	0.64	5668	3247	1992	552	7239	10408
P13	B4	3/13/07 7:35	3/14/07 0:03	16.5	7.55	10.0	1.44	0.27	5736	3226	1902	626	7461	10446
P16	B3	3/14/07 18:31	3/15/07 9:02	14.5	7.55	10.4	1.95	0.72	5289	3570	1947	645	6565	10382
P17	B4	3/15/07 6:32	3/15/07 23:12	16.7	7.55	10.1	1.61	0.32	6044	2334	2058	550	7539	10444
P20	B3	3/16/07 21:17	3/17/07 11:46	14.5	7.55	10.1	1.87	0.73	7097	3633	2550	604	6559	10385
P21	B4	3/17/07 5:33	3/17/07 21:29	16.0	7.55	10.2	1.72	0.32	3515	3686	1246	669	7227	10433
P32	B4	3/22/07 17:00	3/23/07 6:00	13.0	7.53	10.1	2.03	0.30	5310	3204	1854	646	5869	10312
P33	B3	3/22/07 18:18	3/23/07 8:50	14.5	7.55	11.1	2.11	0.68	5228	3174	1841	555	6582	10387
P36	B4	3/24/07 10:49	3/25/07 2:01	15.2	7.52	10.3	1.40	0.30	5485	3397	1819	652	6862	10362
P37	B3	3/24/07 17:52	3/25/07 6:53	13.0	7.54	10.3	1.71	0.67	5504	3172	1910	529	5885	10327
P40	B3	3/26/07 13:58	3/27/07 3:00	13.0	7.52	12.1	1.06	0.69	6462	3110	2523	536	5874	10294
Average		--	--	14.7	7.5	10.6	1.80	0.49	5288	3310	1855	604	6637	10379
+/- 95% CI		--	--	0.7	0.0	0.3	0.17	0.09	403	145	143	25	334	25

Table D9: Statistical comparison of filter performance between 5 and 7.5 gal/ft^2-min (San Jose)

Comparative Statistics	Run Time (h)	Loading Rate (gpm/ft^2)	Terminal HL (ft)	Turbidity (NTU)		2-5 µm particles		5-15 µm particles		Volume Treated (gal/ft^2-run)	Production Capacity (gal/ft^2-
				Influent	Effluent	Influent	Effluent	Influent	Effluent		
Average difference between 5 and 7.5 gpm/ft^2	-14.9	2.5	0.3	0.03	0.02	-309	51	-79	31	-2285	3306
Percent Change from 5 to 7.5 gpm/ft^2	-50%	50%	2%	2%	5%	-6%	2%	-4%	5%	-26%	47%
Is the 7.5 gpm/ft^2 rate statistically worse?[a]	YES	--	--	no	no	no	no	no	no	YES	no
Total Sample Size (Number of Runs)	40	40	40	40	40	40	40	40	40	40	40

[a] Significance level defined as a = 0.05 for two sample hypothesis test (H_0: $m_5 \geq m_{7.5}$; H_A: $m_5 < m_{7.5}$) using *t*-test

[b] Allowable difference and required statistical detection ability defined by CDPH equivalency criteria; % allowable = 0.2 / (FE Turbidity at 5 gpm/ft^2) = 0.2/0.72 = 27.7%

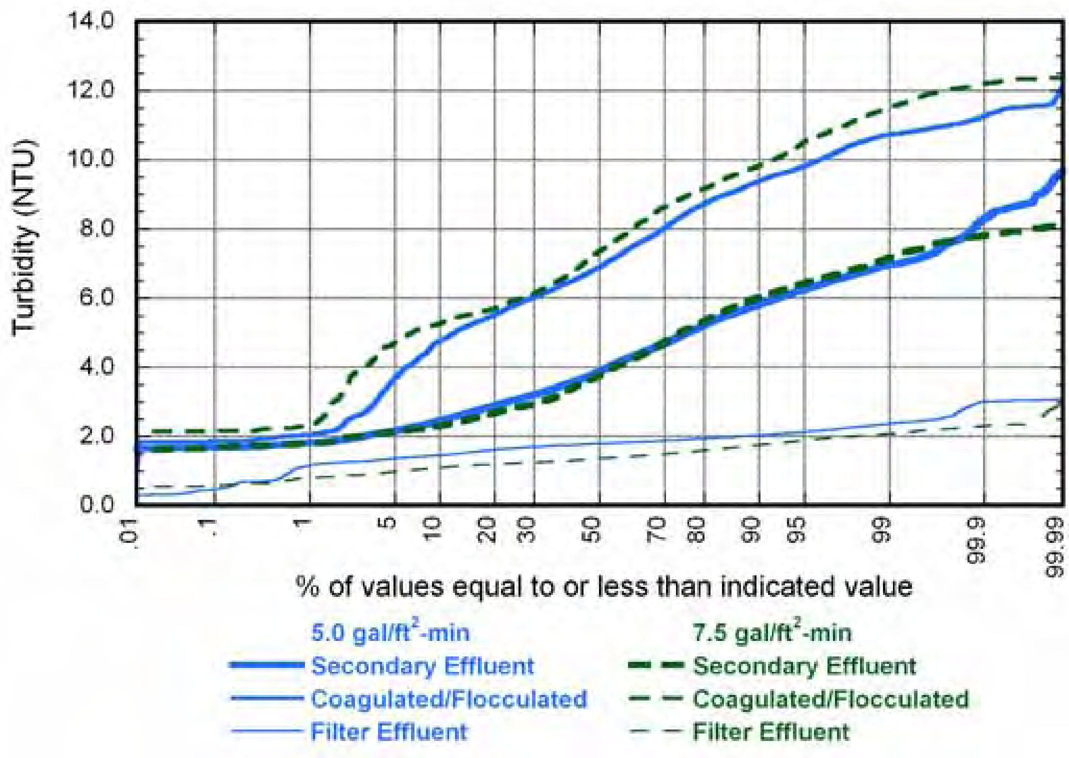


Figure D1. Monterey filter influent and effluent turbidity.

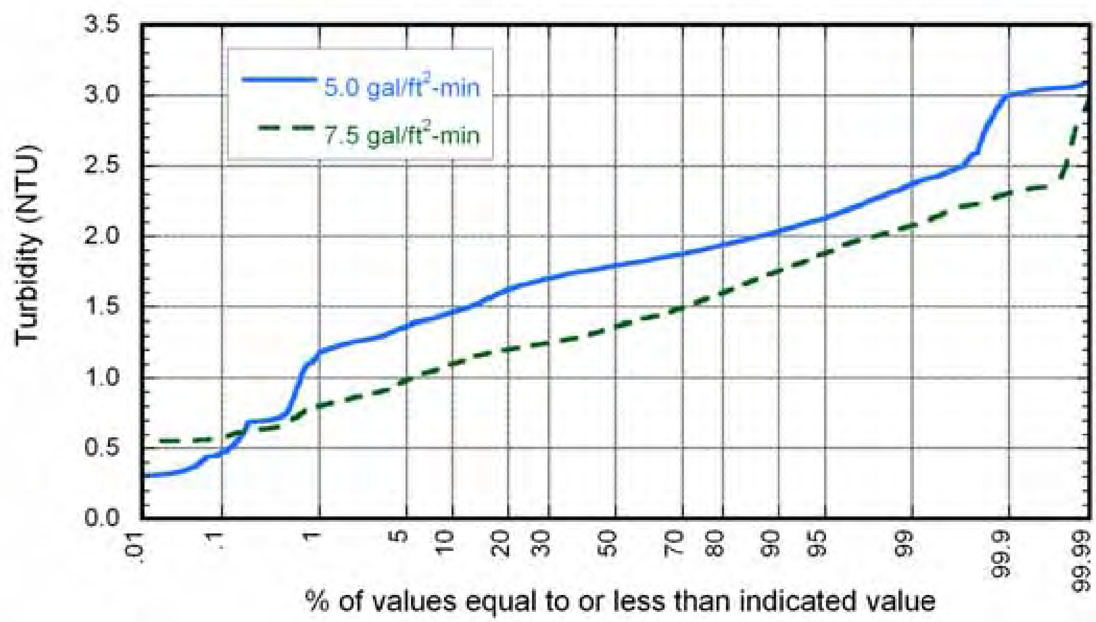


Figure D2. Monterey filter effluent turbidity.

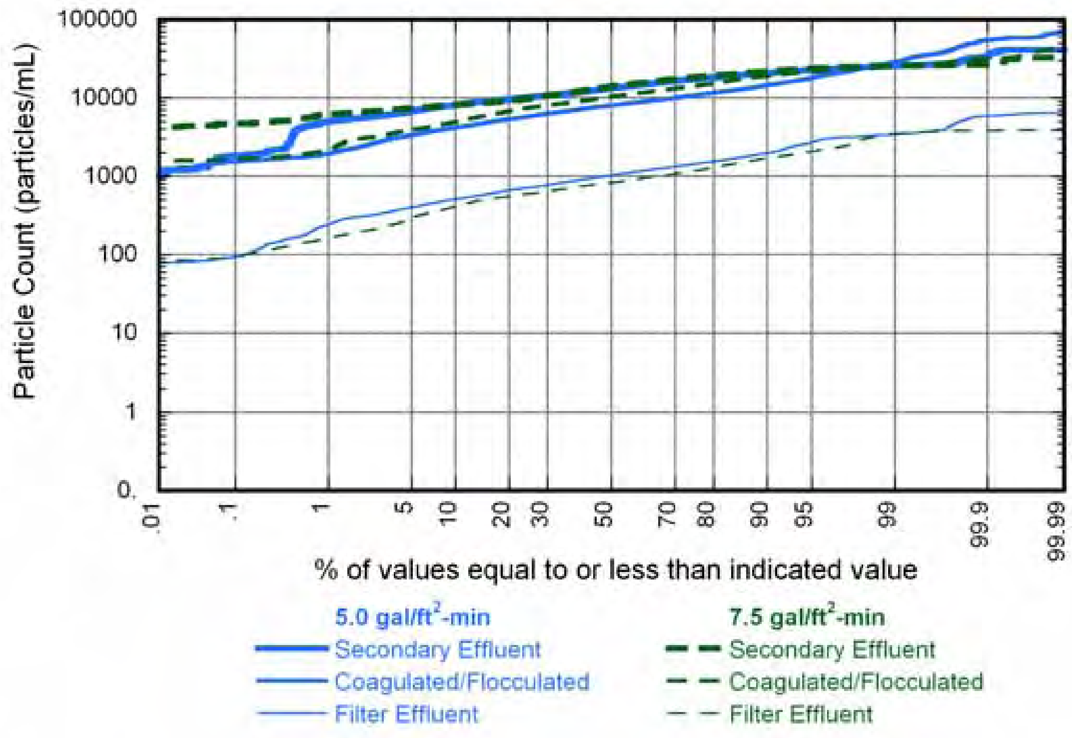


Figure D3. Monterey filter influent and effluent particles (2–5 µm).

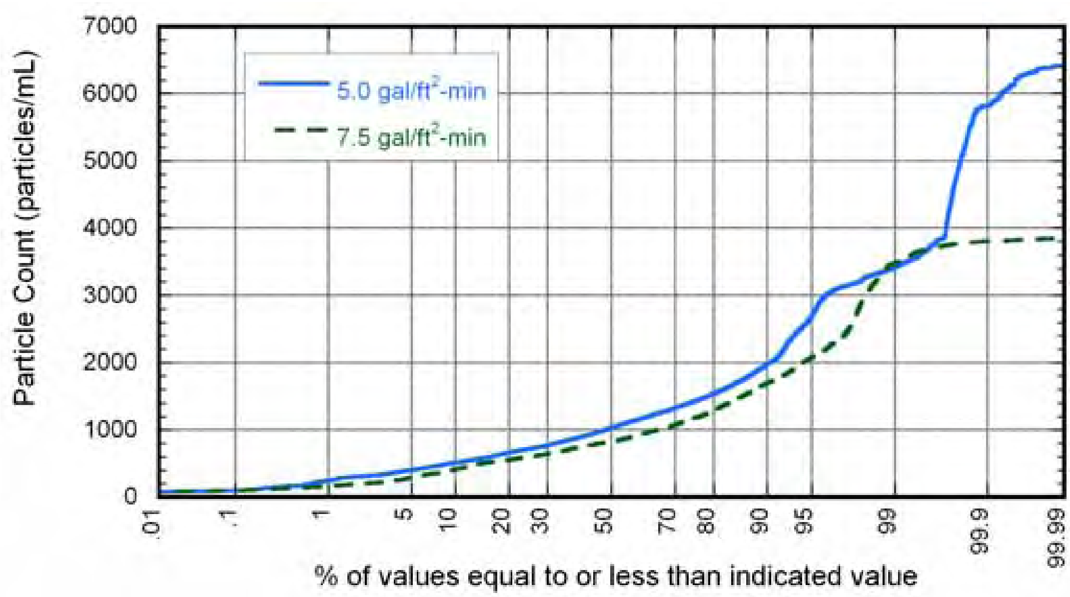


Figure D4. Monterey filter effluent Particles (2–5 µm).

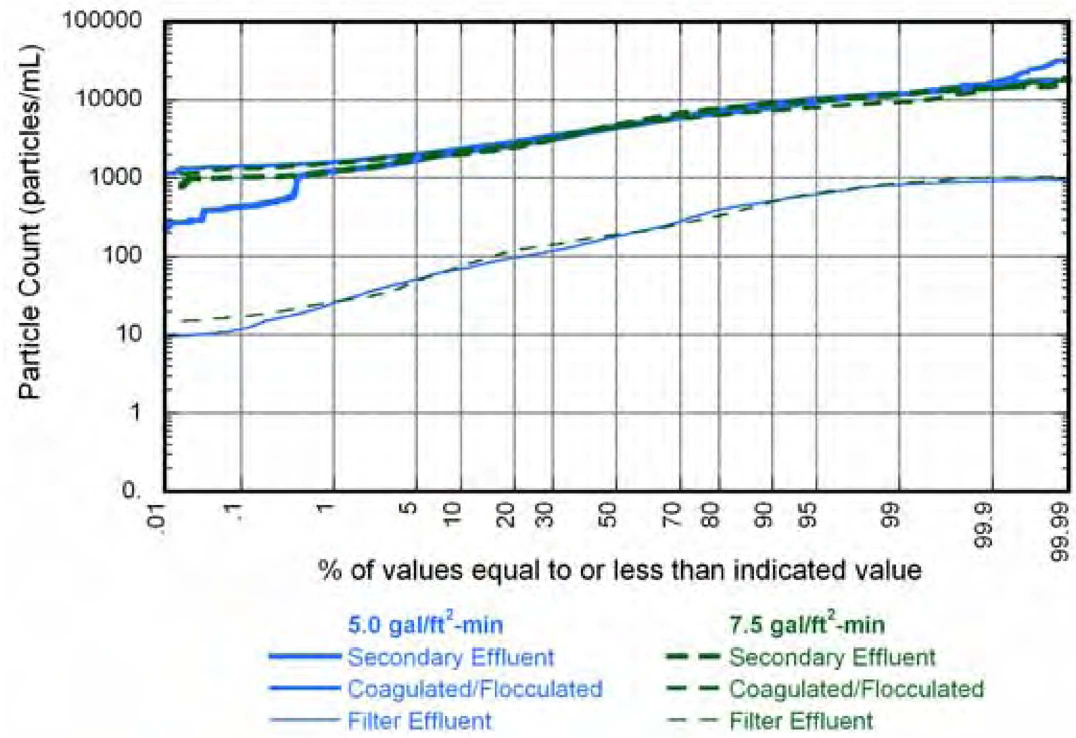


Figure D5. Monterey filter influent and effluent particles (5–15 µm).

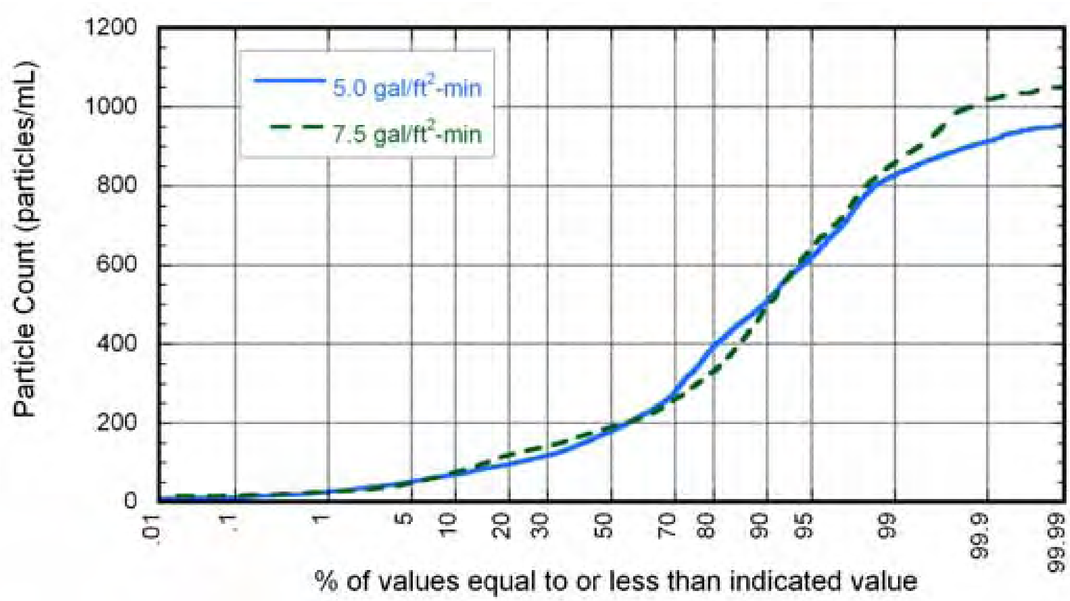


Figure D6. Monterey filter effluent particles (5–15 µm).

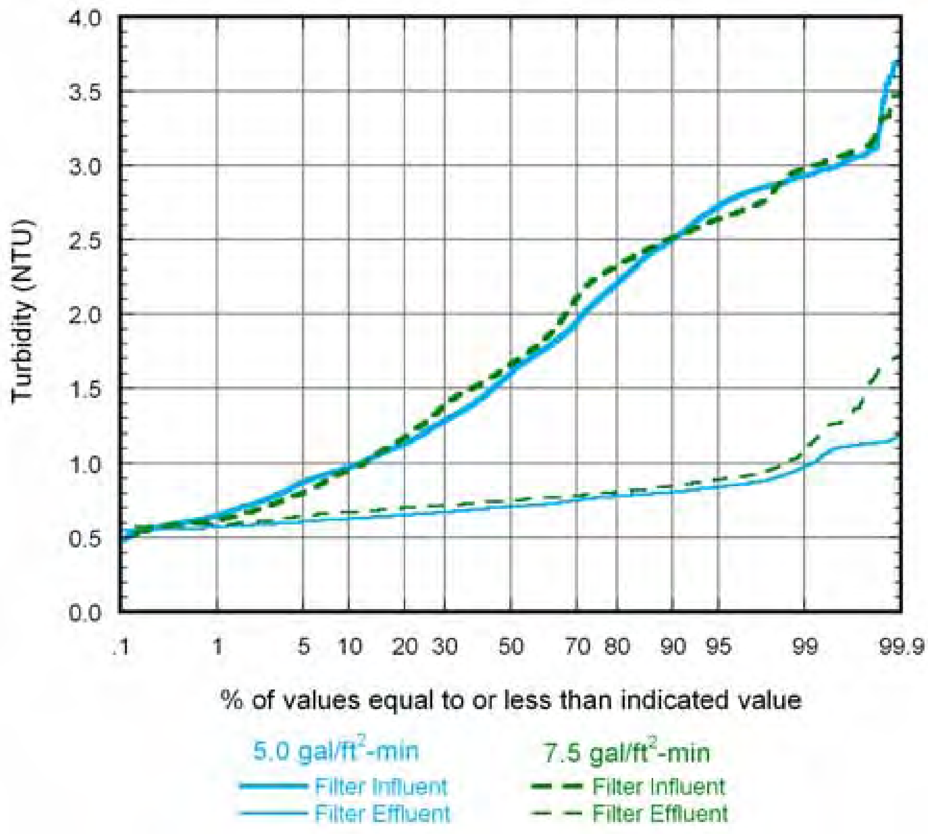


Figure D7. San Jose filter influent and effluent turbidity.

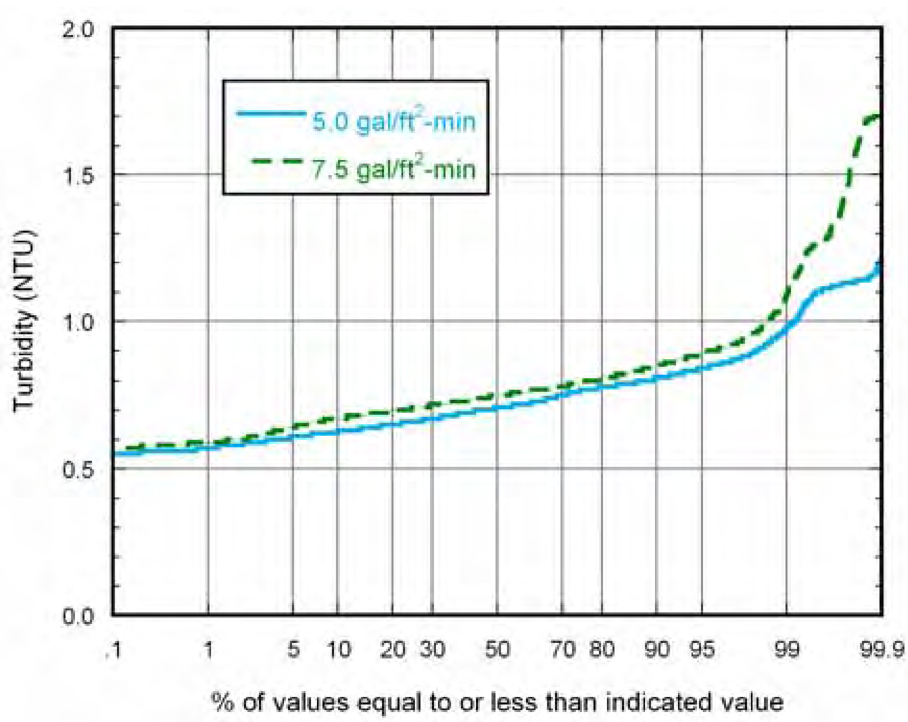


Figure D8. San Jose filter effluent turbidity.

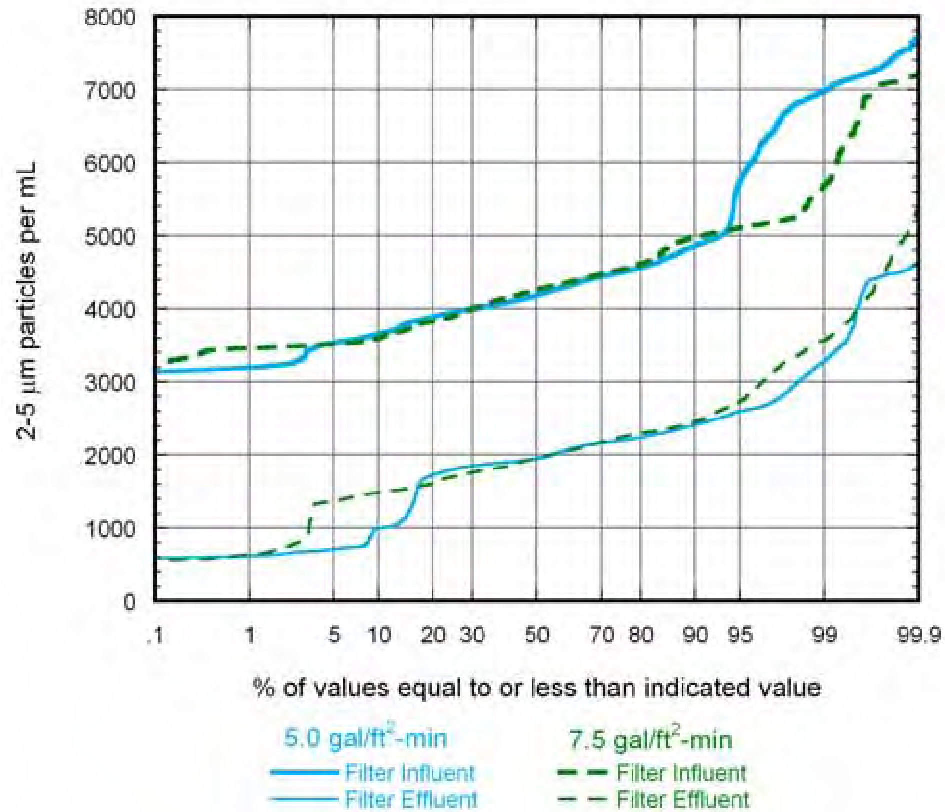


Figure D9. San Jose filter influent and effluent particles (2–5 µm).

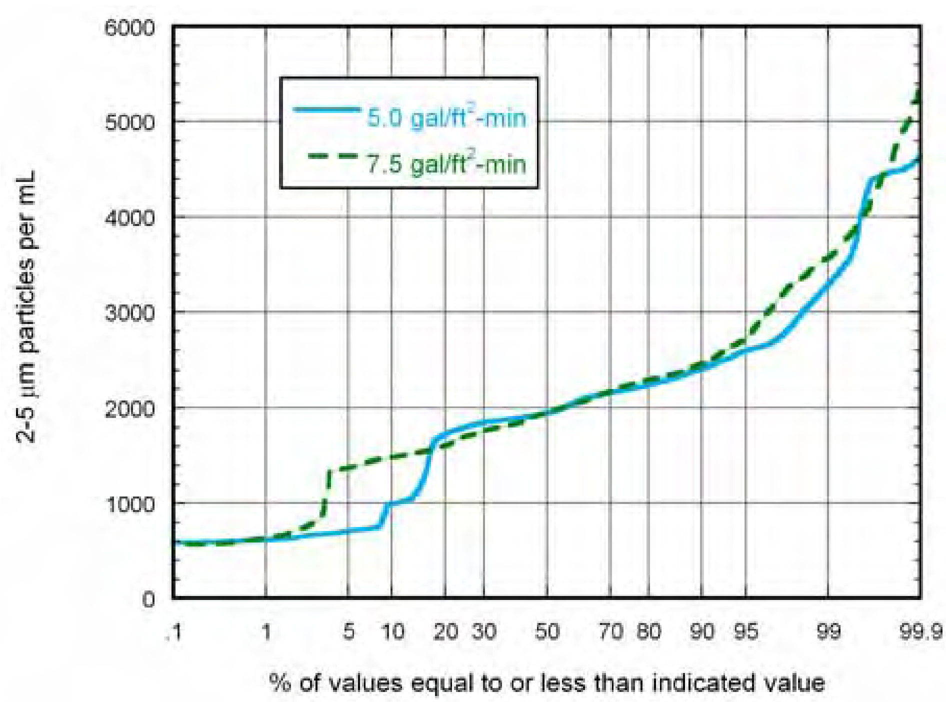


Figure D10. San Jose filter effluent particles (2–5 µm).

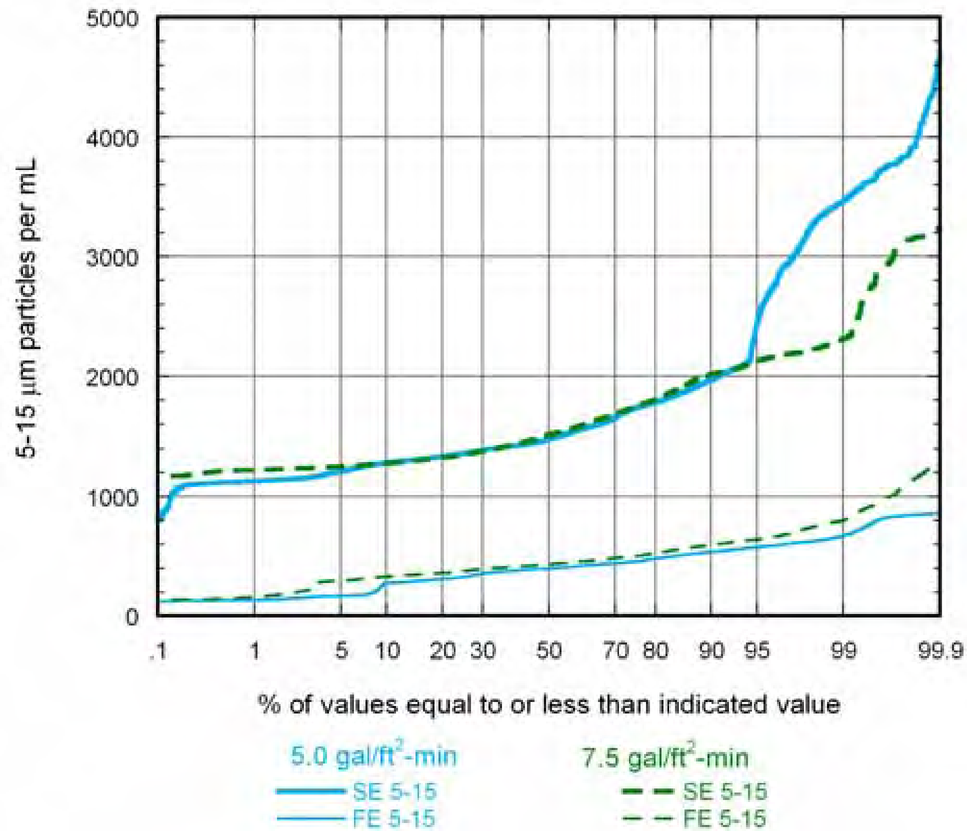


Figure D11. San Jose filter influent and effluent particles (5–15 µm).

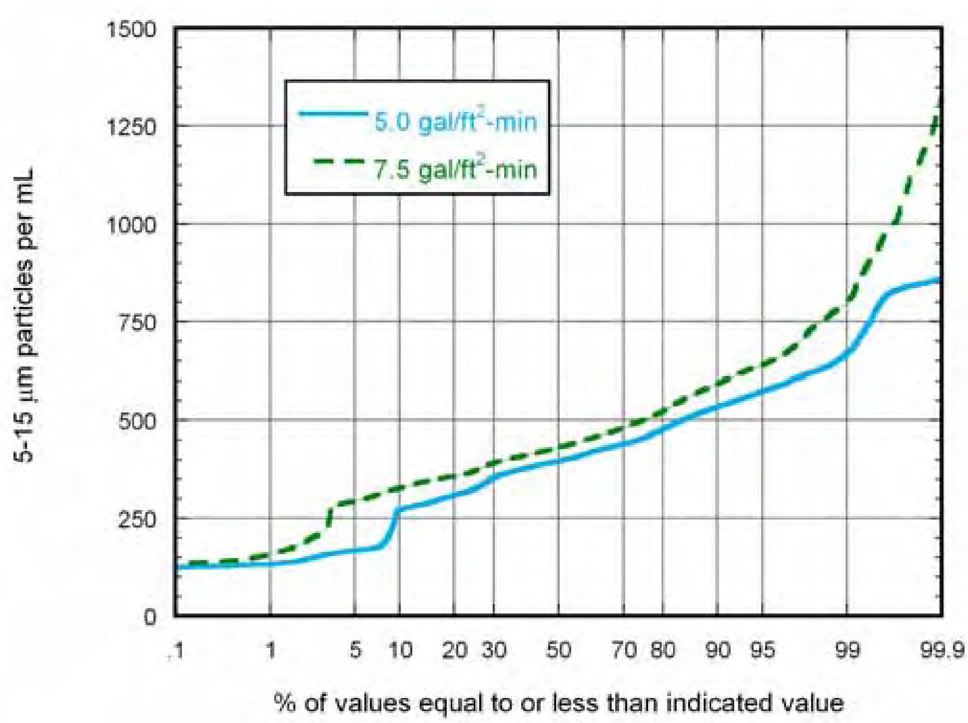


Figure D12. San Jose filter effluent particles (5–15 µm).

APPENDIX E. REGULATORY APPROVAL LETTERS FOR MRWPCA

Summary of correspondence regarding full-scale filter loading rate testing and permanent approval at the Monterey Regional Water Pollution Control Agency (MRWPCA).

3/16/2007 – CDPH letter to Central Coast Regional Water Quality Control Board (CCRWQCB) supporting full-scale filter loading rate testing at MRWPCA.

4/17/2007 – MRWCPA request to CCRWQCB requesting temporary waiver from 5 gal/ft^2-min limit to test loading rates up 7.5 gal/ft^2-min for study.

5/1/2007 – CCRWQCB approval of 12-month waiver to complete loading rate testing.

11/30/07 – CDPH letter approving proposed full-scale equivalency criteria for loading rate comparison.

09/15/2008 – University of California Berkeley (UCB) letter documenting the observed variability in pilot- and full-scale granular media filters.

09/24/2008 – CDPH removing loading rate variability provision from the approval letters for FLEWR participants.

10/2/2008 –UCB to MRWPCA verifying that equivalency criteria were met.

10/8/2008 – MRWPCA request to CDPH for change in allowable loading rate (up to 7.5 gal/ft^2-min).

01/12/2009 – CDPH acceptance of final report and support for permanent change in loading rate requirement for MRWPCA to 7.5 gal/ft^2-min.

Undated – MRWPCA standard operating procedure for operating with two sets of standards for the two loading rate limits.

3/12/2009 – Permanent waiver from CCRWQCB for MRWPCA to operate at filter loading rates up to 7.5 gal/ft^2-min.

State of California-Health and Human Services Agency

Department of Health Services

Northern California Drinking Water Field Operations Branch
Monterey District

California Departmen t of Health

SANDRA SHEWRY
Director

ARNOLD SCHWARZENEGGER Governor

March 16, 2007
System No. 2790002

Mr. Roger W. Briggs, Executive Officer
Central Coast Regional Water Quality Control Board
895 Aerovista Place, Suite 101
San Luis Obispo, CA 93401

Dear Mr. Briggs:

MONTEREY REGIONAL WATER POLLUTION CONTROL AGENCY'S FILTER LOADING EVALUATION FOR WATER REUSE PHASE !I STUDY-WAIVER REQUEST

A research project entitled "Filter Loading Evaluation for Water Reuse" (FLEWR) is currently underway to determine the effect of varying filter loading rates on recycled water quality. Sponsored in part by the National Water Research Institute and the Water Reuse Foundation, the purpose for this research project is to evaluate granular media filter performance at loading rates above the current maximum 5 gpm/ft^2 allowed under the California Water Recycling Criteria, Title 22. Six California Water Pollution Control Agencies, including Monterey Regional Water Pollution Control Agency, which falls under your jurisdiction, are collaborating in this endeavor.

Phase I of this project entailed a pilot study conducted at the Monterey Regional Water Pollution Control Agency to evaluate filter performance at loading rates up to 7.5 gpm/ft^2 and included extensive evaluation of operational parameters and resultant filter performance parameters. Section 60320.5 of the Water Recycling Criteria allows for "other methods of treatment" to those specifically defined in Title 22 provided that the applicant demonstrates to the satisfaction of the California Department of Health Services (CDHS) that the methods of treatment and reliability features will assure an equal degree of treatment and reliability. The equivalency criteria used to evaluate the data from Phase I of this study were:

No significant increase in average turbidity in filter effluent;
No significant increase in average particle counts in the size ranges of 2-5 and 515 microns in filter effluent;
No significant decrease in log removal of MS2 phage; and

iv. No significant decrease in log inactivation of total coliform bacteria through subsequent disinfection.

The Department has reviewed the Phase I findings, and taken into account the equivalency factors listed above. The department finds that under the proper operational conditions, an equivalent degree of filter performance may be achieved to meet Title 22 turbidity performance and virus removal objectives. These operational conditions are listed below as conditions of granting a waiver to the 5 gpm/ft^2 filter loading rate. Waiver of the 5gpm/ft^2 filter loading rate is needed for the study to continue with the full-scale demonstration of higher filter loading rates in Phase II.

1 Lower Ragsdale, Building 1, Suite 120, Monterey, CA 93940-5741
(831) fi55-6939; Fax (831) 655-6944
Internet Address: httpalwww.dhs.ca.qov/ps/ddwem/

Phase II of the research project proposes to evaluate filter performance on a full scale basis at the six participating agencies. Full scale testing would be limited to a minimum number of filters to be determined by the research project Principal Investigators (Pls). The Pls will also outline operational and monitoring controls at each plant to ensure compliance with Title 22 requirements during the study period.

It is our understanding that Monterey Regional Water Pollution Control Agency will be contacting your agency to request a temporary waiver from their current filter loading limitation to conduct Phase II of this research project. The Department recommends that your agency grant such a waiver under the following conditions:

a. Duration of Waiver - twelve months.

b. Maximum plant flow not to exceed 38.5 MUD and Individual filters loading rate not to exceed 7.5 g pm/ ft^2.

c. Individual filter turbidities not to be exceeded:
   I. An average of 2.0 NTU within 24-hour period;
   It. 2.5 NTU for more than 5% of time; and
   ill. 5 NTU at any time.

d. Maintain a minimum CT of 1200 mg/L-min. with a minimum 90 minute modal contact time.

e. Particle count monitoring on individual filter and combined filter effluents.

f. An Interim Operational Plan on the Phase II study addressing: study duration; maximum flow; filter loading rates; filter runs; backwash rate; chemicals (coagulant/polymer) dose, chlorine dose and contact time; reliability provisions; turbidity monitoring frequency; particle count monitoring frequency; filter media inspection frequency. The plan should be submitted for review and approval by the Central Coast Regional Water Quality Control Board (RB) and CDHS prior to initiating the study.

9• Upon completion of the study, submit a detailed report on the study findings to the RB and CDHS.

If you have any questions regarding the waiver conditions, please contact me at (510) 620-3452, or Ms. Jan Sweigert, Monterey District Engineer, at (831) 655-6939.

Sincerely,

Catherine S. Ma, P.E., Chief
North Coastal Region
Drinking Water Field Operations Branch

Copy:
Matthew Keeling - Central Coast RWQCB - 895 Aerovista Place, Suite 101, San Luis Obispo, CA 93401
Robert Holden - Monterey Regional Water Pollution Control Agency - 5 Harris Court, Building D, Monterey, CA 93940
Bahman Sheikh, Ph.D., P.E. - Water Reuse Consultant, 3524 22nd Street, San Francisco, CA 94114-3406 Monterey County
Environmental Health Department - 1270 Natividad Road, Salinas, CA 93906

COPY RTP file

Monterey Regional Water Pollution Control Agency

"Dedicated to meeting the wastewater and reclamation needs of our member agencies, while protecting the environment."

Treatment Facility and Water Recycling Project:
Mailing Address: P.O. Box 1790, Marina, CA 93933-1790
Shipping Address Only: 14811 Del Monte Blvd., Marina CA 93933
(831) 883-1118 or 424-1108, FAX: (831) 883-0516
Website: www.mrwpca.org

April 17, 2007

Mr. Matthew Keeling
California Regional Water Quality Control Board
Central Coast Region
895 Aerovista Place, Suite 101
San Luis Obispo, CA 93401

SUBJECT: Filter Loading Evaluation for Water Reuse

Dear Mr. Keeling:

Dr. Bahman Sheikh, Dr. Kara Nelson, Dr. Jim Crook, and Dr. Bob Cooper have been leading a research study since 2003 about filter loading rates. This research project has been funded by the WateReuse Foundation, the National Water Research Institute, Monterey Regional Water Pollution Control Agency, Monterey County Water Resources Agency, City of Santa Rosa, and the Sanitation Districts of Los Angeles County. The goal is to increase the capacity to produce recycled water from media filters. Phase 1 of the study was conducted through October 2005 and involved the testing with pilot filters. That work was very successful in showing that the filter loading rate could be raised by 50% above the Title 22 limit of 5 gpm/ft^2 with no decrease in pathogen or particle removal. California Department of Health Services (DHS) sent the enclosed letter to you on March 16, 2007 recommending that we proceed to Phase 2 of the research project. Phase 2 will involve using the full-scale (Salinas Valley Reclamation Project) filters to test the safety and effectiveness of the higher filter loading rate.

The Interim Operation Plan for the Phase 2 work:

- Study duration: one year though we hope to gather all the data we need within a few months.
- Maximum flow: we will not exceed 38.5 MGD instantaneous flow through the tertiary treatment system.
- Filter Loading Rates: We will set at least one filter for a 7.5 gpm/ft^2 rate (90+% of the time the flow should be within +/- 0.2 gpm/ft^2 of the set point. The actual instantaneous value should not exceed 7.8 gpm/ft^2).
- Filter runs: We will run the filters until the headloss is excessive or until the turbidity increases. We will perform as many runs as necessary to provide statistically significant results or as needed to determine information needed for the scientific part of the study.
- Backwash rate: We currently over clean the filters during backwash we expect we will still be over cleaning at the higher loading rate. Consequently, we will leave the backwash rate as is. If the clean bed headloss increases significantly or if the backwash water turbidity increases significantly, then we will re-optimize backwashes.
- Chemical (coagulant/polymer) dose: We expect that the chemical dosage will need to be higher for the higher loading rate but well within the range of our equipment. We will maintain our real time manual adjustment of dosage by our on-site operator. The operators will increase dosage if turbidity is too high and reduce dosage it if it is too low.
- Chlorine dose and contact time: We will continue providing a disinfection CT of 1,200 mg/L-minutes. If we have any problem maintaining the median of total coliform below 2.2 MPN/100 mL then we will increase chlorine dosage. We will maintain a minimum calculated modal contact time

Joint Powers Authority Member Entities:
Boronda County Sanitation District, Castroville Service Area 14, County of Monterey, Del Rey Oaks, Fort Ord, Marina Coast Water District, Monterey, Moss Landing County Sanitation District, Pacific Grove, Salinas, Sand City, and Seaside

of at least 90 minutes.

- Reliability provisions: We have addressed the reliability issues in our Engineering Report. Please contact us with questions or for a copy of the report.
- Turbidity monitoring frequency: Turbidity is monitored continuously at each filter and at the combined filtered flow. We record turbidities at each location each minute.
- Particle counter monitoring frequency: We will have a particle counter on the secondary effluent (not required by DHS but important for the scientific part of the project), the test filter, and the combined filtered effluent. The particle counters record counts every minute.
- Filter media inspection frequency: We visually inspect filters that are in service daily. We will conduct a more thorough inspection should the clean bed headloss increase significantly and not be reduced through backwash optimization. Otherwise, we will conduct in depth filter inspections after the conclusion of this research project.

This Operation Plan has been provided to DHS as part of the Engineering Report.

DHS recommends that you approve a waiver to our current permit to allow a loading rate of 7.5 gpm/ft^2 with duration of at least one year. We ask that you grant such a waiver. Thank you in advance.

If I can provide any additional information, please let me know.

Sincerely,

James A. Dix

James Dix
WWTP Operations Manager and Chief Operator
jamesd@mrwpca.com

Enclosure
cc: Catherine S. Ma, Chief DHS
Cheryl Sandoval, Monterey County Environmental Health

California Regional Water Quality Control Board
Central Coast Region

895 Aerovista Place, Suite 101, San Luis Obispo, California 93401-7906
(805) 549-3147 • Fax (805) 543-0397
http://www.waterboards.ca.gov/centralcoast

Linda S. Adams.
Secretary for Environmental Protection

Arnold Schwarzenegger
Governor

May 1, 2007

Mr. James Dix
Monterey Regional Water Pollution Control Agency
P.O. Box 1790
Marina, CA 93933

Dear Mr. Dix:

RE: FILTER LOADING EVALUATION FOR WATER REUSE – WAIVER REQUEST; MONTEREY REGIONAL WATER POLLUTION CONTROL AGENCY, MONTEREY COUNTY

We reviewed your April 17, 2007, letter requesting our approval of phase 2 of the Filter Loading Evaluation for Water Reuse (FLEWR) at your facility that entails a waiver of the filter loading rate established pursuant to Title 22[1] and by reference in your Water Reclamation Requirements Order no. 94-82[2]. We are also in receipt of the Department of Health Services (DHS) March 16, 2007, recommendation to approve a waiver of the filter loading rate given various conditions are met as outlined in their letter.

We have no objection to the proposed phase 2 FLEWR study given all conditions outlined in the DHS recommendation letter are met and the study is conducted in accordance with the Interim Operation Plan outlined in your letter.

If you have questions regarding this matter, please contact **Matthew Keeling at (805) 549-3685** or mkeeling@waterboards.ca.gov, or John Robertson at (805) 542-4630.

Sincerely,

FOR Roger W. Briggs
Executive Officer

See next page for list of cc's

[1] California Code of Regulations, Title 22, Division 4, Chapter 3, Article 1, Section 60301.320
[2] Paragraph (Prohibition) A.3

California Environmental Protection Agency

Recycled Paper

State of California—Health and Human Services Agency
California Department of Public Health

MARK B HORTON, MD, MSPH
Director

ARNOLD SCHWARZENEGGER
Governor

November 30, 2007

Dr. Bahman Sheikh
Water Reuse Consultant
3524 22nd Street
San Francisco, CA 94114-3406

Dear Dr. Sheikh:

FILTER LOADING RATE EVALUATION FOR WATER REUSE PHASE II STUDY

The research project "Filter Loading Evaluation for Water Reuse (FLEWR)" is currently underway to determine the effect of varying filter loading rates on recycled water quality. Its objective is to evaluate granular media filter performance at filter loading rates above the current maximum of 5 gpm/ft^2 allowed under the California Water Recycling Criteria (WRC), California Code of Regulations, Title 22. Section 60320.5 of the WRC allows other methods of treatment than those specifically stated in the WRC, if the recycled water agency demonstrates to the satisfaction of this department that the other methods of treatment and reliability features will assure an equal degree of treatment and reliability. Phase I of this project entailed a pilot study conducted at the Monterey Regional Water Pollution Control Agency (MRWPCA) to evaluate filter performance at filter loading rates up to 7.5 gpm/ft^2. MRWPCA has completed Phase I which successfully demonstrated treatment equivalency and has proceeded into Phase II of the study involving full-scale treatment demonstration at 5 gpm/ft^2 and at 7.5 gpm/ft^2 under controlled conditions.

On October 10, 2007, the FLEWR Principal Investigation Team (PI) met with the California Department of Public Health (CDPH) and made a presentation of their findings of Phase I and presented proposals for determining treatment equivalency during Phase II. Subsequently, you and I had a telephone conversation regarding the Phase II proposals. The major points of that discussion are listed below:

1. Phase II Equivalency Criteria are:
    a. No significant increase in mean turbidity of filter effluent;
    b. No significant increase in mean concentration of 2-5 and 5-15 μm particles in filter effluent;
    c. The definition for "significant" in above two bullets as proposed in Slide 22 of the presentation is acceptable to CDPH.

Division of Drinking Water and Environmental Management
P.O. Box 997377, MS 7400, 1616 Capitol Avenue, 2nd Floor, Sacramento, CA 95899-7377
(916) 449-5577 (916) 449-5575 Fax
Internet Address: www.cdph.ca.gov

    d. No significant decrease in ability to disinfect filter effluent.

2. Dr. George Tchobanoglous' conceptual proposal to demonstrate that no significant decrease in the ability to disinfect filter effluent is acceptable. Details of the proposal need to be provided.
3. Approval under Section 60320.5 of the WRC for higher filter loading rates will be given on a case-by-case basis for each individual recycled water treatment plant.
4. Those recycled water agencies seeking approval for higher filter loading rates must demonstrate equivalency between performance at 5 gpm/ft^2 and 7.5 gpm/ft^2 under optimized conditions at both filter loading rates.

Both the Santa Rosa Subregional Wastewater Treatment Plant and Los Angeles County Sanitation District's San Jose Creek Water Reclamation Plant have submitted to CDPH a proposed engineering report as well as a request for a temporary waiver of the WRC filter loading rate requirement. CDPH's review has found these proposals to be inadequate in providing information on characterizing the existing plant operation and performance for a direct comparison of the impacts on the effluent quality at higher filter loading rates.

On November 6, 2007, Ms. Kara Nelson submitted the "FLEWR Phase II Protocol: Virus Disinfectability" for CDPH review to address item 2 above. The protocol was developed to assess the impacts of filter loading rate on the ability to disinfect viruses in the filter effluents and the experiment as proposed will be included at all full-scale facilities participating in the Phase II Study.

CDPH has the following recommendations and comments that should be addressed in the engineering reports or as amendments to the FLEWR Study Phase II and/or FLEWR Phase II Protocol: Virus Disinfectability:

1. Please provide the final list of the Phase II participating recycled water agencies and name of the treatment facility. Indicate if the facility is seeking approval for the higher filter loading rate or providing data on the performance of a full-scale facility at the higher filter loading rate.

2. Adequate characterization on the existing operation and performance of the treatment processes at the participating recycled water agency is needed to establish a baseline for direct comparison of the impacts on the filtered and disinfected effluents at higher filter loading rates. Specifically, as a minimum, the monthly averages and the ranges (min – max) of chemical, physical and bacteriological quality for the raw, secondary, tertiary and disinfected effluents for the past 12 months should be provided in the engineering report.

3. Those plants that are currently operating at less than 5 gpm/ft^2, must demonstrate and establish a similar operation and performance baseline of their treatment process at 5 gpm/ft^2 in addition to that of their existing filter loading rate.

4. The participating recycled water agency should describe in their engineering report how the plant is being optimized currently at the existing filter loading rate, as well as how they will be optimized at the higher filter loading rate(s) (i.e., 5 and/or 7.5 gpm/ft^2).

5. The batch MS2 phage disinfection experiment should include one additional chlorine dose that corresponds to the existing CT of those participating recycled water agencies that are currently providing less than 450 mg-min CT in their existing chlorination-disinfection process.

6. Monitoring of physical, chemical and microbiological parameters should be concurrent whenever feasible to allow for proper comparison as well as correlation of effluent quality and performance parameters at different filter loading rates. If multiple filters are operating at the target filter loading rates of current, 5 and/or 7.5 gpm/ft^2, one filter must be designated to be the "test filter" at each of the filter loading rates at any given time for the purposes of data collection and subsequent performance evaluation. The "test filter" for each filter loading rate must have a minimum duration of one week to ensure that appropriate and adequate data can be collected to allow for evaluating impacts on filter run cycles and backwash processes at the target filter loading rates.

7. The samples used for the batch virus disinfectivity experiments should be taken from the "test filter" under controlled conditions such that they are representative of the treated effluent produced at each of the target filter loading rate(s). The samples should be adequately identified and characterized with the critical water quality parameters associated with the designated test filter including but not limited to: turbidity, particle counts, coliform bacteria, UVT (for UV disinfection only) and chlorine residuals.

8. Data analysis should be performed and presented in the form of probability distribution plots including but not limited to: turbidity, particle counts, virus log inactivation, and overlaid for comparison of treatment performance distributions for the different filter loading rates employed in the study.

Dr. Bahman Sheikh
Page 4
November 30, 2007

CDPH strongly supports and endorses the research efforts of the FLEWR Study. We look forward to working with the PI and the participating recycled water agencies in the furtherance of our understanding on the impacts of filter loading rate as well as the efficacy and effectiveness of recycled water treatment processes on pathogen removal and inactivation.

If you have any questions concerning this matter, please feel free to contact me at (916) 449-5577.

Sincerely,

Gary H Yamamoto

Gary H. Yamamoto, P.E., Assistant Chief
Division of Drinking Water
and Environmental Management

UNIVERSITY OF CALIFORNIA, BERKELEY

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Gordon Williams, Doctoral Candidate
Department of Civil & Environmental Engineering
University of California
Berkeley, CA 94720-1712
Tel./Fax 510-643-9714
gordon@berkeley.edu

September 15, 2008

Jeffery L. Stone, Chief
Technical Operations Branch- Recycled Water Unit
California Department of Public Health
1180 Eugenia Place, Suite 200
Carpinteria, CA 93013-2000

Dear Jeff,

I am writing in response to the CDPH recommended condition for participants of the Filter Loading Evaluation for Water Reuse (FLEWR) study, which specifies the allowable exceedance of the 7.5 gpm/ft^2 filter loading rate. The specific condition appeared in the following two CDPH letters to Regional Water Boards:

> 1) In the February 28th, 2008 letter from Catherine Ma (CDPH) to Robert Klamt (North Coast Regional Water Quality Control Board) regarding the City of Santa Rosa's waiver to participate, in condition (b):
>
> > *"Maximum plant flow not to exceed 66.9 MGD, and individual filter loading rate* ***not to exceed 7.5 gpm/ft^2 95% of the time****, with instantaneous rates not to exceed 8.0 gpm/ft^2."*
>
> 2) In the June 19th, 2008 letter from Catherine Ma (CDPH) to Bruce Wolfe (San Francisco Bay Regional Water Board), regarding the Delta Diablo Sanitation District's (DDSD) waiver to participate, condition (b):
>
> > *"Maximum plant flow not to exceed 16.2 MGD (based on chlorine contact time limitations), and individual filter loading rate* ***not to exceed 7.5 gpm/ft^2 95% of the time****, with instantaneous rates not to exceed 8.0 gpm/ft^2."*

It is my understanding that this recommendation is intended to allow plants to use 7.5 gpm/ft^2 as a flow rate target or "set-point", while allowing for the inherent variability in the instantaneous rate due to limitations of the flow controllers. It is also my understanding that a loading rate that rounds to 7.5 gal/ft^2-min is within this limit (e.g. 7.549 is not considered to exceed 7.5). Unfortunately, neither DDSD nor the City of Santa Rosa can control the rate precisely enough to operate at a *set-point* of 7.5 gal/ft^2-min, while not exceeding 7.5 gal/ft^2-min 95% of the time. For these facilities to meet this 95% limit, they would need to operate at a lower set-point, corresponding to a lower average loading rate. In this case, it would prevent the FLEWR study from comparing filter performance at the same two rates (5.0 and 7.5 gal/ft^2-min) at all facilities.

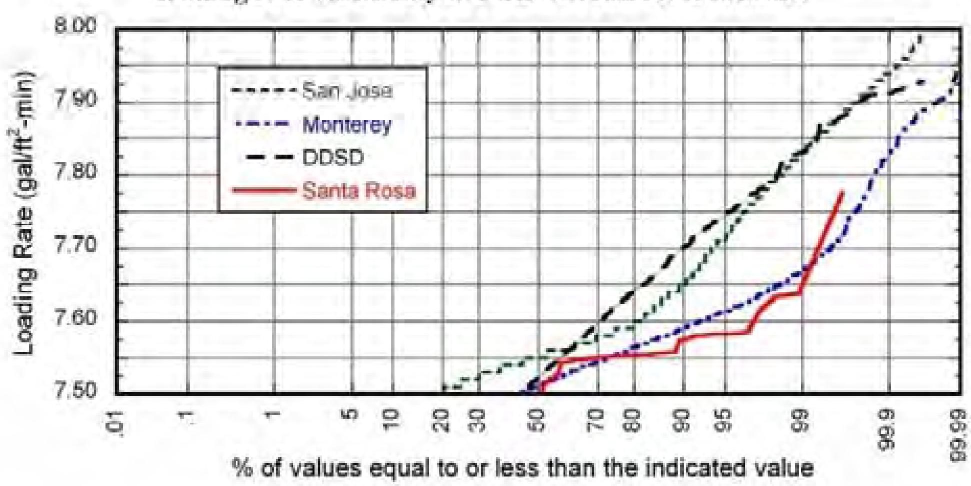
Two other FLEWR participants, the City of San Jose and the Monterey Regional Water Pollution Control Agency (MRWPCA), were not subject to this "95% of the time" condition[1,2] and have completed their full-scale filter testing. When testing the 7.5 gal/ft^2-min set-point, these plants were above 7.5 gal/ft^2-min for 56 and 27% of the time, respectively (Table 1 and Figure 1). Nevertheless, they were both able to meet the CDPH equivalency criteria for producing equivalent water.

Table 1: Filter loading rate control ability at FLEWR participants

Phase II Participant	Loading Rate Control Point	% of time >7.5 gal/ft^2-min average (min-max for individual runs)	Rate not exceeded 95% of time (gal/ft^2-min)
MRWPCA	Filter effluent valve	27 (16-32)%	7.6
City of San Jose	Filter effluent valve	56 (15-74)%	7.7
DDSD	Tertiary influent pump	44%*	7.8*
City of Santa Rosa	Tertiary influent pump	39%*	7.6*

*Indicates predicted value

Figure 1: Actual (Monterey and San Jose) and predicted (DDSD and Santa Rosa) loading rate variability at FLEWR full-scale facilities



I have predicted the loading rate variability for DDSD and Santa Rosa using sample data from these plants (Figure 1) and expect the variability at these plants will be similar to those observed at Monterey and San Jose. For DDSD and Santa Rosa to stay below 7.5 gal/ft^2-min 95% of the time, their average loading rate would need to be significantly less than 7.5 gal/ft^2-min (either 7.2

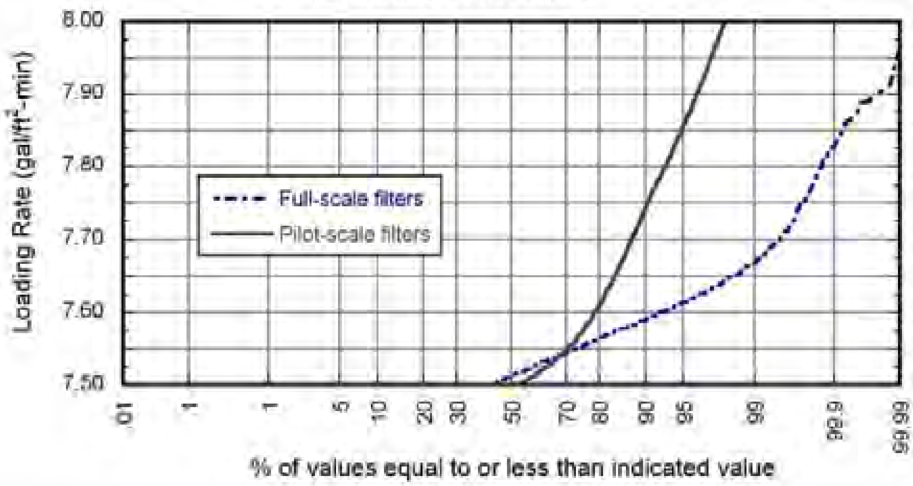
[1] **San Jose**: Used bay discharge filters for FLEWR testing which were identical to their water reclamation filters, except they were not subject to the Title-22 loading rate restrictions.

[2] **Monterey**: Regarding loading rate limitations, from November 28, 2006 letter from Jan Sweigert (CDPH) to Robert Holden. "The Department hereby approves the Engineering Report with the following comments…The Department recognizes the objectives of the Phase II Study is to demonstrate the filter performance with a loading rate of 7.5 gpm/ft^2. The maximum filter loading rate for any individual filter, during the course of this study period, must not exceed 8.0 gpm/ft^2 at any time and the maximum flow must not exceed 38.5 MGD within 24 hours.

or 7.3 gal/ft^2-min). On the other hand, if they are allowed to test at a set-point of 7.5 gal/ft^2-min, I expected that the loading rate will be at or below 7.8 and 7.6 gal/ft^2-min 95% of the time, respectively (Table 1).

For an additional point of reference, you can look at data from the FLEWR pilot tests, where the variability was greater than at the full-scale plants (Figure 2). Even with the loading rate exceeding 7.5 gal/ft^2-min 30% of the time and 7.9 gal/ft^2-min 5% of the time, the study found equivalent filter performance could be achieved at both test rates, 5.0 and 7.5 gal/ft^2-min.

Figure 2: Comparison between the pilot and full-scale filter loading rate variability at Monterey



These variations in the loading rate, both at the pilot and the full-scale, have not hindered the demonstration that equivalent filter performance can be achieved at higher loading rates. Requiring plants to stay below 7.5 gal/ft^2-min 95% of the time would prevent them from testing an actual set-point of 7.5 gal/ft^2-min. On behalf of the FLEWR team and participants, I request that the "95% of the time" condition be dropped and the CDPH recommendations be modified to only limit the filter loading *set-point* and the maximum instantaneous rate, as follows:

> *Maximum plant flow not to exceed… MGD, and individual filter loading rate* ***set-point*** *not to exceed 7.5 gpm/ft^2, with instantaneous rates not to exceed 8.0 gpm/ft^2.*

This will ensure that the participants can set their target loading rate at 7.5 gal/ft^2-min, and data comparisons between plants will be at the same loading rate. Plants will operate with the minimum amount of loading rate variability possible, as it will provide the best data for the study.

Regards,

Gordon Williams

MARK B HORTON, MD, MSPH
Director

State of California—Health and Human Services Agency
California Department of Public Health

ARNOLD SCHWARZENEGGER
Governor

September 24, 2008

Mr. Gordon Williams
Department of Civil and Environmental Engineering
University of California
Berkeley, CA 94720-1712

Subject: FLEWR Study

Dear Gordon:

This letter is in response to your letter dated September 15, 2008 concerning allowable exceedance of the 7.5 gpm/ft^2 filter loading rate. The Department established the allowable exceedance at 95% of the time based on what we believed would be reasonably achievable for the treatment plants being evaluated as part of this study. As you have indicated, flow control is not precise enough at the Delta Diablo and Santa Rosa plants to ensure meeting the 95% limitation without reducing plant flow rates below the targeted study loading rate of 7.5 gpm/ft^2. You further note that other plants involved in this study were not previously subjected to this limitation. The Department has evaluated your request and hereby withdraws the 95% limitation previously imposed by letters dated February 28, 2008 to the North Coast RWQCB (for Santa Rosa) and June 19, 2008 to the San Francisco Bay RWQCB (for Delta Diablo) during the course of the FLEWR demonstration study. All other conditions however remain unchanged.

Any future CDPH consideration of long-term increased filter loading rates at any facilities will likely take into account the establishment of the 95% limitation along with any necessary facility modifications to achieve this objective.

If you have any questions, please call this office at (805) 566-9767.

Sincerely,

Original signed by:

Jeffrey L. Stone, Chief
Recycled Water Unit
CDPH-DWTOB

Technical Operations Section – Recycled Water Unit
1180 Eugenia Place, Suite 200 Carpinteria, CA 93013-2000
(805) 566-9767; (805) 745-8196 fax
Internet Address: http://www.cdph.ca.gov/programs/Pages/DWP.aspx

cc: North Coast RWQCB – Ms. Katherine Kuhlman
San Francisco Bay RWQCB – Mr. Vince Christian
City of Santa Rosa – Mr. Dan Carlson
Delta Diablo Sanitation District – Ms. Amanda Roe

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October 2, 2008

Mr. Robert Holden
Principal Engineer
5 Harris Court, Building D
Monterey, CA 93940-5756

RE: Completion of full-scale FLEWR testing at MRWPCA

Dear Bob,

The FLEWR project team has been performing full-scale filter tests at the MRWPCA treatment plant over the past 18 months as part of the Filter Loading Evaluation of Water Reuse (FLEWR) study. The purpose of the FLEWR study is to improve our understanding of the effects filter loading rates higher than 5 gal/ft^2-min (the maximum loading rate allowable under the California Water Recycling Criteria) have on granular media filter performance. The full-scale portion of the study (Phase II) was designed to compare filter performance at 7.5 and 5 gal/ft^2-min. As previously agreed upon with the California Department of Public Health (CDPH), the results from each plant participating in the study will be evaluated using a set of criteria to determine if the filters operated at both rates receive an equal degree of treatment. The equivalency criteria for Phase II of the project are as follows:

Phase II Equivalency Criteria[1]

1. No significant* increase in mean turbidity of filter effluent;
2. No significant* increase in mean concentration of 2-5 and 5-15 μm particles in filter effluent;
3. No significant decrease in the ability to disinfect filter effluent

*Where significant increase[2] = $\dfrac{0.2\ \text{NTU}}{\text{NTU produced at 5.0 gal/ft}^2\text{-min}}$ (reported as percent)

After completing our analysis, we can report that by optimizing the coagulant dose specific to the loading rate at 7.5 gal/ft^2-min and adjusting the target filter effluent turbidity to be 0.4 NTU lower than turbidity at 5 gal/ft^2-min, all of the above equivalency criteria were met. Sixty-two filter runs were completed (31 at each loading rate) spanning the three seasons that MRWPCA produces

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[1] Adapted from November 30, 2007 letter from Gary Yamamoto (CDPH) to Bahman Sheikh (FLEWR PI)
[2] For MRWPCA, significant increase is defined at 11.3% (0.2 NTU/1.78 NTU).

reclaimed water, and the necessary statistically rigors of the FLEWR study were achieved[3]. The following is a summary of our findings, as they pertain to the equivalency criteria.

Turbidity and Particle Concentration Metrics

Turbidity and particle counts were monitored and recorded every minute for the secondary effluent, coagulated/flocculated filter influent[4], and the test filter effluent. The mean value for each parameter was determined for each run (Appendix A) and the overall mean for each rate was then calculated (Tables 1 and 2). Statistical analyzes were performed on the mean values for each loading rate. Probability plots of turbidity and particle counts (2-5 and 5-15 μm ranges) provide additional detailed comparisons (Appendix B).

Turbidity and Particle Counts

A comparison of the overall performance of filters operated at the two different loading rates is provided in Figure 1. The average filter effluent turbidity and the average particle counts in the 2-5 μm range were lower at the 7.5 gal/ft^2-min rate than at the 5.0 gal/ft^2-min rate. The average particle counts in the 5-15 μm range were equal at the two loading rates. A more detailed description of our analysis is provided in the subsequent paragraphs.

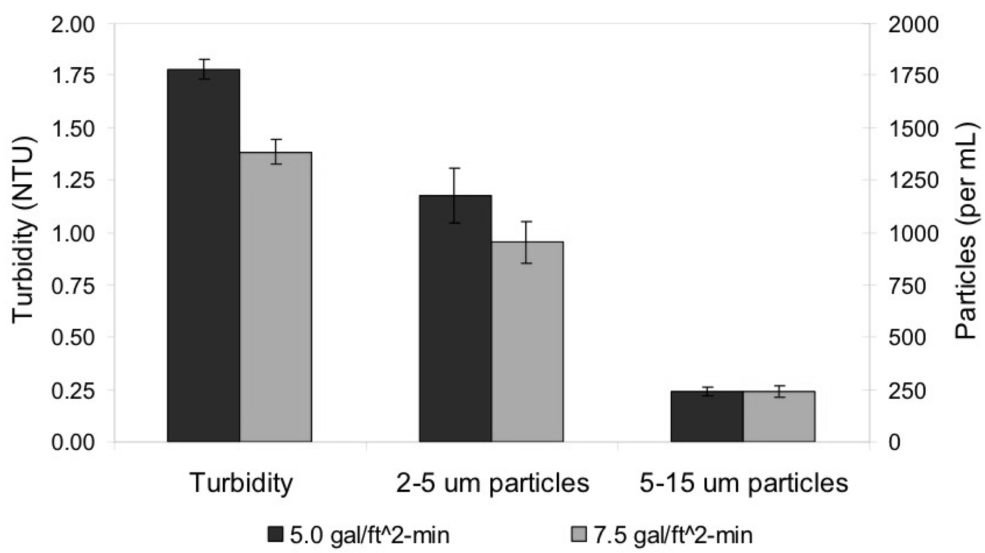

Figure 1: Comparison of particle concentration metrics in MRWPCA filter effluent.
Error bars indicate the 95% confidence interval for the plotted mean value

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[3] The least significant difference was less than or equal to the equivalency criteria definition of significant (i.e. the least significant difference for all equivalency parameters was ≤ 11.3%; See Appendix A)

[4] Coagulated/flocculated monitoring equipment was installed after the preliminary test period and became operational prior to Run 2 (9/11/2007)

Even though it was only possible to test one loading rate at a time, the secondary effluent water quality during the FLEWR testing was statistically equivalent for the two loading rates. The mean secondary effluent turbidity was slightly higher during 5 gal/ft^2-min testing, but the secondary effluent particle counts were lower for the 5 gal/ft^2-min rate (Tables 1 and 2).

Table 1: MRWPCA filter run characteristics and turbidity results*

Loading Rate (gal/ft^2-min)	Total Number of Runs (n)	Run Time (h)	Coagulant Dose (mg/L)	Turbidity (NTU)		
				Secondary	*Filter Influent*	*Filter Effluent*
5.0	31	22.6 ± 0.73	5.1 ± 0.6	4.05 ± 0.22	7.00 ± 0.39	1.78 ± 0.05
7.5	31	12.0 ± 0.61	7.7 ± 0.8	4.00 ± 0.30	7.41 ± 0.43	1.38 ± 0.06
%Δ from 5.0-7.5	--	**-47%**	**51%**	-1%	6%	**-22%**

*Significant differences (α=0.05) marked in bold; values given are *mean ± 95% CI*

A 51% increase in coagulant usage was needed to achieve equivalent performance when operating at 7.5 gal/ft^2-min. The flocculation process reduced the overall number of particles, as many smaller diameter particles agglomerated to form larger particles. The higher coagulant doses at 7.5 gal/ft^2-min caused a significant increase in 2-5 µm particles and turbidity in the filter influent, as compared to 5 gal/ft^2-min doses, but this shift also led to an increase in the overall filter performance in terms of particle removal through the filter at the higher rate (Figure 1).

The improved filter performance at the 7.5 gal/ft^2-min rate (versus the 5.0 gal/ft^2-min rate) is reflected in the difference in filter effluent turbidities. The filter effluent turbidity at 7.5 gal/ft^2-min *decreased* by 22% (or 0.4 NTU; Table 1). This higher performance also led to a 19% (or 220 particles/mL) *decrease* in the number of 2-5 µm particles in the filter effluent (Table 2). The mean number of particles in the 5-15 µm size range was the same for both loading rates (Table 2). All three of these data sets are well below the 11.3% increase defined as significant by the above equivalency criteria.

Table 2: MRWPCA particle counter results from 2-5 and 5-15 µm size ranges*

Loading Rate (gal/ft^2-min)	Particles 2-5 µm (mL^{-1})			Particles 5-15 µm (mL^{-1})		
	Secondary	*Filter Influent*	*Filter Effluent*	*Secondary*	*Filter Influent*	*Filter Effluent*
5.0	13700 ± 1200	8900 ± 1100	1170 ± 130	4990 ± 540	4960 ± 450	239 ± 20
7.5	14400 ± 1200	11000 ± 1600	950 ± 100	5230 ± 560	4700 ± 400	239 ± 26
%Δ from 5.0-7.5	4.6%	**24%**	**-18.9%**	4.7%	-5.3%	0%

*Significant differences (α=0.05) marked in bold; values given are *mean ± 95% CI*

Preliminary Test Period

An initial series of 10 filter runs was completed in Summer 2007, during which operators optimized the coagulant dose such that the filter effluent would remain around 1.8 NTU at both loading rates. However, under these conditions the effluent particle counts (both 2-5 and 5-15 µm ranges) were higher when the filters were operated at 7.5 versus 5 gal/ft^2-min (Appendix C). The coagulation strategy was then changed for FLEWR testing, such that operators optimized the coagulant dose to produce a lower filter effluent turbidity for the 7.5 versus the 5 gal/ft^2-min rate. Since these preliminary runs were performed under different operating conditions from the rest of the MRWPCA testing, they were not used in the analysis to determine equivalency. However, it is important to note that even if these runs were included in the overall analysis, the equivalency criteria and the statistical rigors of the FLEWR study would still be met.

Aborted Runs

Seven runs could not be included in the analysis for various reasons. These reasons included instrumentation malfunction, operator error, and plant upsets (Appendix D). Because only one loading rate could be tested at a time, the impact of these events was specific to one loading rate, and these data were not used in the overall analysis to avoid biasing the results.

Assessment of Disinfectability

The ability to disinfect the filter effluent, as required by the third equivalency criterion, was tested by performing 73 batch coliform disinfection experiments at the MRWPCA laboratory (protocol in Appendix E) and the virus disinfection experiment at UC Berkeley as specified by the Virus Disinfectability protocol[5]. No decrease in the ability to disinfect the water was found. The following is a summary of the disinfection findings (detailed results reported in Appendix F).

Coliform disinfection ability

The bench scale disinfection testing was designed to mimic the full-scale disinfection. Since MRWPCA practices pre-chlorination prior to tertiary filtration, coliform concentrations going into the filters were typically low. Filter effluent samples were collected and immediately taken to the laboratory for the disinfection experiment. Sodium hypochlorite was added to the samples such that the residual at 120 min was approximately 10 mg-Cl/L (a 1200 mg/L-min C*T target). The actual mean CT values were slightly higher than 1200 mg/l-min, due to variation in the chlorine residual (Table 3).

In terms of total coliform bacteria disinfection, all samples except those specified in Table 3 footnotes b-c had a most probable number (MPN) less than the 2.2 per 100 mL of sample. The CDPH Water Recycling Criteria specify the 7-day *median* concentration be less than 2.2 MPN/100 mL. Therefore, both rates had adequate disinfection in terms of total coliform bacteria. In addition, 22 and 11% of the disinfection samples tested at 5 and 7.5 gal/ft^2-min, respectively, had coliform concentrations of 1-2 MPN/100 mL. Overall, there was no reduction in the ability to disinfect total coliform bacteria at the higher loading rate.

[5] Submitted to by Professor Kara Nelson to CDPH on November 6, 2007

Table 3: MRWPCA results from bench scale disinfections tests

Loading Rate (gal/ft^2-min)	# of Bench Scale Tests	Avg C*T (mg/L-min)	Total Coliform Bacteria		*E. Coli*	
			Samples with positive well[a]	$\geq$2.2 MPN per 100 mL	Samples with positive well[a]	$\geq$2.2 MPN per 100 mL
5.0	37	1281	10	2[b]	1	0
7.5	37	1338	5	1[c]	1	0

[a] IDEXX has a detection limit of 1 MPN/100 mL (one positive well)

[b] For loading rate 5 gal/ft^2-min: Run 11 (10/1/2007) at 35 MPN/100mL and Run 12 (10/3/2007) at 6 MPN/100mL

[c] For loading rate 7.5 gal/ft^2-min: Run 7 (9/24/2007) at 18 MPN/100mL

In terms of *E. coli*, only one sample at each filter loading rate had any positive wells (both were 1.0 MPN/100mL[6]; Table 3), thus indicating that there was no decrease in the disinfection of *E. coli* at the 7.5 gal/ft^2-min filter loading rate.

Virus disinfection ability

The UC Berkeley virus disinfection protocol used filter effluent from MRWPCA treated at both loading rates. Several chlorine doses were applied to filter effluent samples, and inactivation of seeded MS2 coliphage was measured after 90 min (Table 4). The dose-inactivation rates (slopes of curves in Figure 2) for water treated at both loading rates were similar and not statistically different (α = 0.05). These slopes act an indicator of the disinfection potential of the tertiary effluent, and thus there was no decrease in the ability to disinfect MS2 virus seeded into the filter effluent at the higher loading rate.

Table 4: Results from Virus Disinfection Study at MRWPCA

Loading Rate (gal/ft^2-min)	Target CT (mg/L-min)	Chlorine Dose (mg/L)	Chlorine Residual (mg/L)[a]	CT Value (mg/L-min)	Log $[MS2]_{INITIAL}$[b]	Log $[MS2]_{FINAL}$[b,c]	Log MS2 Inactivation
5	450	6	5.0	450	7.2	6.9 $\pm$ 0.1	0.3
	1200	14	14.1	1300		6.8 $\pm$ 0.2	0.5
	2400	27	25.9	2300		6.2 $\pm$ 0.1	1.0
7.5	450	6	5.5	490	7.2	7.0 $\pm$ 0.1	0.2
	1200	14	13.6	1200		6.8 $\pm$ 0.3	0.5
	2400	27	24.5	2200		6.1 $\pm$ 0.1	1.1

[a] After 90-min contact time for all samples

[b] Units are log(PFU/mL)

[c] Mean log concentration $\pm$ 95% confidence on mean

[6] Even though the IDEXX method has been shown to have a significant incidence of false-positives for *E. coli.* Yakub, GP et al. (2002). Evaluation of Colilert and Enterolert defined substrate methodology for wastewater applications. *Water Environment Research* 74 (2), pp.131-135.

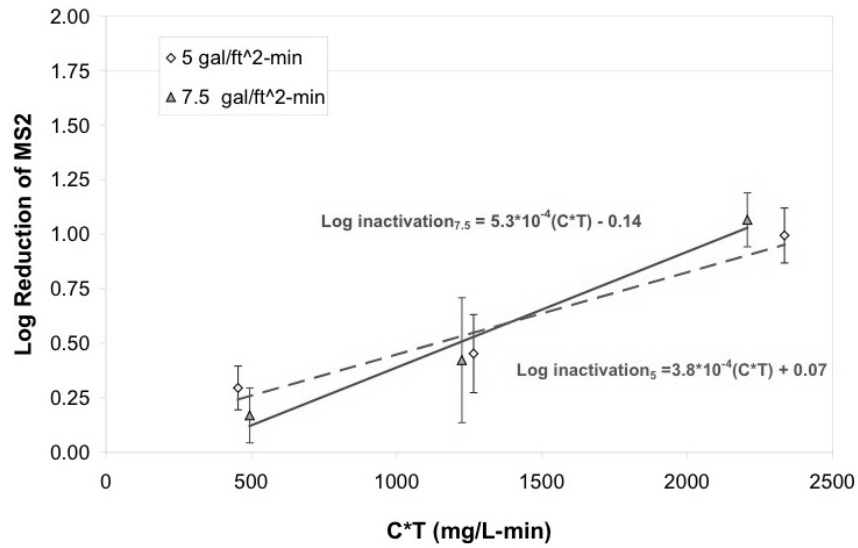

Figure 2: **Virus disinfection results for Monterey**
The log inactivation of seeded MS2 coliphage vs CT value. No decrease in the ability to disinfect MS2 coliphage was detected between the two filter loading rates. Linear regression lines are shown for both rates (solid green/dark line = 7.5 gal/ft^2-min; dashed blue/light line = 5 gal/ft^2-min).

Conclusions

In terms of the performance metrics (equivalency criteria) agreed upon by CDPH, your plant achieved equivalent performance while testing at a 7.5 gal/ft^2-min filter loading rate, as compared to operations at 5 gal/ft^2-min. To accomplish equivalent particle counts in the 5-15 μm size range, it was necessary to use a coagulant dose that was about 50% higher at 7.5 gal/ft^2-min than at 5 gal/ft^2-min. No difference in disinfection ability was detected through the bench-scale disinfection experiments.

We hope that this analysis is sufficient to support your request for a permanent waiver from the CDPH. Please contact us if you have any questions or need additional information.

Sincerely,

Kara L. Nelson

cc: Bahman Shiekh, James Crook, Robert Cooper

Monterey Regional Water Pollution Control Agency

"Dedicated to meeting the wastewater and recycled water needs of our member agencies, while protecting the environment."

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Website: www.mrwpca.org

October 8, 2008

Mr. Gary Yamamoto, P.E., Acting Chief
Division of Drinking Water and Environmental Management
California Department of Public Health, MS 7400
P. O. Box 997377
Sacramento, CA 95899-7377

Dear Mr. Yamamoto:

The research project entitled "Filter Loading Evaluation for Water Reuse" (FLEWR) has been underway for the past five years to determine the effect of varying filter loading rates on recycled water quality. Sponsored in part by the National Water Research Institute and the Water Reuse Foundation, the purpose for this research project was to evaluate granular media filter performance at loading rates above the current maximum 5 gpm/ft^2 allowed under the California Water Recycling Criteria, Title 22.

Monterey Regional Water Pollution Control Agency (MRWPCA) successfully proved equivalency through the pilot (Phase 1) testing in October 2005. Catherine Ma, of your Department, provided a letter dated March 16, 2007 to the Central Coast Regional Water Quality Control Board (CCRWQCB) supporting a temporary waiver allowing MRWPCA to operate tertiary filters up to 7.5 gpm/ ft^2. The CCRWQCB issued such a waiver on May 1, 2007. MRWPCA successfully completed the full-scale (Phase 2) testing in August 2008. UC Berkeley's letter, tabulations, and graphics attached to this letter confirm that the equivalency criteria established by your Department have been met. I request your concurrence with the project team's conclusion regarding the equivalency of water quality at the 7.5 gpm/ ft^2 rate with that at the 5 gpm/ft^2 rate. I further request that you recommend to the CCRWQCB that they grant MRWPCA a permanent waiver from the 5 gpm/ft^2 filter loading rate.

Please contact Bob Holden, 831-645-4634 should you wish to set up either a telephone conference or a meeting between our staffs by mid November to discuss the permanent waiver.

Regards,

Keith Israel

Keith Israel, General Manager

attachment

cc: Bob Hultquist, Catherine Ma, Jan Sweigert, Jeff Stone, Bahman Sheikh, Kara Nelson, Jim Crook, Bob Cooper, Gordon Williams, Tom Kouretas, James Dix, Bob Holden, Curtis Weeks, and Roger Briggs

Joint Powers Authority Member Entities:
Boronda County Sanitation District, Castroville Water District, County of Monterey, Del Rey Oaks, Fort Ord, Marina Coast Water District, Monterey, Moss Landing County Sanitation District, Pacific Grove, Salinas, Sand City, and Seaside

State of California—Health and Human Services Agency
California Department of Public Health

MARK B HORTON, MD, MSPH
Director

ARNOLD SCHWARZENEGGER
Governor

January 12, 2009

Mr. Roger Briggs
Executive Officer
Central Coast Regional Water Quality Control Board
895 Aerovista Place, Suite 101
San Luis Obispo, CA 93401

Gentlemen:

MONTEREY REGIONAL WATER POLLUTION CONTROL AGENCY – REQUEST FOR APPROVAL TO OPERATE AT AN INCREASED FILTER LOADING RATE

The California Department of Public Health (CDPH) is in receipt of a letter from the Monterey Regional Water Pollution Control Agency (MRWPCA) dated October 8, 2008, in which a request was made for granting a permanent waiver to operate at filter loading rates of up to 7.5 gpm/ft^2. Current Water Recycling Criteria, California Code of Regulations (CCR) Title 22, prescribe a maximum allowable filter loading rate of 5 gpm/ft^2 for granular media type filters. However, CDPH notes that under Section 60320.5, CCR Title 22, allows for alternatives provided that it can be demonstrated that such alternatives provide an equal degree of treatment and reliability. Although MRWPCA filed the request for a permanent waiver to operate at higher loading rates than prescribed by CCR Title 22, CDPH considers such a request as being approval to operate at higher rates, under CCR Title 22 Section 60320.5. The MRWPCA request is based upon an 18-month full scale demonstration study conducted at the MRWPCA wastewater treatment plant as part of the Filter Loading Evaluation of Water Reuse (FLEWR) study being carried out at various locations throughout the state. The letter of request was accompanied by a summary report, dated October 2, 2008, submitted by Dr. Kara Nelson of University of California, Berkeley, on behalf of the FLEWR Project Investigation Team.

The study findings at the MRWPCA facility have verified that equivalent filter performance can be achieved at the higher filter loading rate under previously agreed upon equivalency criteria which include:

1. No significant increase in mean turbidity of filter effluent;

2. No significant increase in mean concentration of 2-5 and 5-15 micron particles in filter effluent; and

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3. No significant decrease in the ability to disinfect filter effluent.

$$\text{Where significant increase} = \frac{0.2\ \text{NTU}}{\text{NTU produced at } 5.0\ \text{gpm/ft}^2} \quad \text{(reported as percent).}$$

For MRWPCA, significant increase is defined at 11.3% (0.2 NTU/1.78 NTU).

CDPH notes that equivalent filter performance was achieved by increasing the coagulant dose to roughly double the current rate, and by maintaining an average effluent turbidity of 1.5 NTU. In order to ensure this level of performance, it will be necessary for the facility to continue this practice when operating at the higher flow rate. With this in mind, CDPH recommends that the Central Coast Regional Water Quality Control Board grant approval through your permitting process to MRWPCA to operate its filters at loading rates up to 7.5 gpm/ft^2, subject to the following requirements:

1. Maximum loading rate for each filter shall not exceed 7.5 gpm/ft^2 at any time.

2. At loading rates above 5 gpm/ft^2, coagulant dose shall be optimized in the same manner as was practiced during the demonstration study to minimize effluent turbidity levels. This will require evaluating filter influent turbidity at all times.

3. Combined filter effluent turbidity shall not exceed any of the following:
   -An average of 1.5 NTU within a 24-hour period;
   - 2.5 NTU more than 5 percent of the time within a 24-hour period; and
   -5 NTU at any time.

4. Continuous turbidity monitoring of each individual filter shall be conducted.

5. Turbidity performance compliance shall be determined using the levels of recorded turbidity taken at intervals of no more than1.2 hours over a 24-hour period.

6. To insure that the conditions contained in this letter are met effectively and efficiently, MRWPCA shall draft and implement a Standard Operating Procedure (SOP) for operating the tertiary plant at filter loading rates of up to 5 gpm/ft^2 and at rates greater than 5 gpm/ft^2 (up to 7.5 gpm/ft^2). The SOP shall list the conditions in the high filter loading rate alternative and provide instructions on the necessary steps and/or adjustments to be done to ensure the conditions are met during high loading rate filter operation. This SOP shall be available in the control room at all times. In addition, control room operators shall receive training on

operating the tertiary filters using the high loading rate SOP by the Chief Plant Operator (CPO). The training shall be developed by the CPO with assistance from the FLEWR Project Investigation Team.

If you have any questions concerning this letter, please contact Mr. Jeffrey Stone at (805) 566-9767.

Sincerely,

Gary H. Yamamoto

Gary H. Yamamoto, P.E., Chief
Division of Drinking Water
and Environmental Management

cc: Keith Israelk
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Rich Mills
Division of Financial Assistance
State Water Resources Control Board
P.O. Box 944212
Sacramento, CA 94244-2130

SOP for Tertiary Filter Operation at Loading Rates > 5 gpm/ft^2

Overview:

- In January 2009 the MRWPCA received approval to operate the SVRP tertiary filters at loading rates up to 7.5 gpm/ft^2, subject to the limits on combined turbidity and maximum filter loading rate specified below. These "new" limits only apply at filter loading rates greater than 5 gpm/ft^2 (high-rate mode).

- When the tertiary filters are operated at or below the Title 22 maximum loading rate of 5 gpm/ft^2 (low-rate mode) the existing turbidity and loading rate limits (also listed below) apply.

- This SOP provides directions on how to run the tertiary filters in low-rate or high-rate mode

Filter Operation:

- **High-rate mode** – To operate a filter (or filters) at a higher (greater than 5 gpm/ft^2) loading rate, the control room operator will increase the hydraulic loading limit for the filter(s) from 5 gpm/ft^2 to 7.5 gpm/ft^2 as follows:
    1. Click on the Filter Control button
    2. Click on the Limits button
    3. At the Operational Limit input box, change the setting from 5 gpm/ft^2 to 7.5 gpm/ft^2

- **Limits at higher loading rates (between 5 gpm/ft^2 and 7.5 gpm/ft^2):**
    1. Maximum loading rate for each filter shall not exceed 7.5 gpm/ft^2 (7.8 mgd) at any time
    2. Combined filter turbidity daily average: ≤ 1.5 NTU
    3. Daily maximum turbidity: ≤ 5 NTU
    4. Percentage of time above 2.5 NTU during a 24-hour period**:** ≤ 5%
    5. Peak filter hydraulic loading: ≤ 7.5 gpm/ft^2 (7.8 mgd) per filter

- **Low-rate mode** – To operate a filter or filter(s) at 5 gpm/ft^2-or less, the control room operator will reduce the hydraulic loading limit to each filter from 7.5 gpm/ft^2 to 5.1 gpm/ft^2 as follows:
    1. Click on the Filter Control button
    2. Click on the Limits button
    3. At the Operational Limit input box, change the setting from 7.5 gpm/ft^2 to 5.1 gpm/ft^2

- **Limits at 5 gpm/ft^2 or below (low-rate mode):**
    1. Combined filter turbidity daily average: ≤ 2.0 NTU
    2. Daily maximum turbidity: ≤ 5 NTU
    3. Peak filter hydraulic loading: ≤ 5.45 gpm/ft^2 (5.65 mgd) per filter

California Regional Water Quality Control Board
Central Coast Region

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Linda S. Adams.
Secretary for
Environmental Protection

Arnold Schwarzenegger
Governor

March 12, 2009

Mr. James Dix
WWTP Operations Manager
Monterey Regional Water Pollution Control Agency
5 Harris Court, Building D
Monterey, CA 93940

Dear Mr. Dix:

RE: REQUEST FOR APPROVAL TO OPERATE AT AN INCREASED FILTER LOADING RATE – WATER RECLAMATION REQUIREMENTS ORDER 94-82; MONTEREY REGIONAL WATER POLLUTION CONTROL AGENCY (SALINAS VALLEY RECLAMATION PROJECT), MONTEREY COUNTY

We reviewed the January 12, 2009 letter from the California Department of Public Health (CDPH) recommending our approval of an increased filter loading rate of 7.5 gallons per minute per square foot (gpm/ft^2) for the Salinas Valley Reclamation Project tertiary filters. The CDPH recommendation is based on an 18-month, full-scale demonstration project showing equivalent filter performance at the increased filter loading rate. We are also in receipt of your Standard Operating Procedures [for Tertiary Filter Operation at Loading Rates > 5 gpm/ft^2] submitted on March 4, 2009, pursuant to requirement number six of the January 12, 2009, CDPH recommendation letter.

We have no objection to the operation of the Salinas Valley Reclamation Project tertiary filters at a loading rate of 7.5 gpm/ft^2 given you implement all of the requirements outlined in the January 12, 2009, CDPH recommendation letter. Our approval of the increased filter loading rate entails a waiver of the filter loading rate established pursuant to Title 22[1] and by reference in your Water Reclamation Requirements Order no. 94-82[2]. This letter serves as our authorization to operate the tertiary filters at the higher loading rate, subject to CDPH requirements, until such time as Water Reclamation Requirements Order No. 94-82 for your facility is updated to reflect this change.

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[1] California Code of Regulations, Title 22, Division 4, Chapter 3, Article 1, Section 60301.320
[2] Paragraph (Prohibition) A.3

California Environmental Protection Agency

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If you have questions regarding this matter, please contact **Matthew Keeling at (805) 549-3685** or mkeeling@waterboards.ca.gov, or Burton Chadwick at (805) 542-4786.

Sincerely,

Harvey Packard

For Roger W. Briggs
Executive Officer

cc:

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California Environmental Protection Agency

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Advancing the Science of Water Reuse and Desalination




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