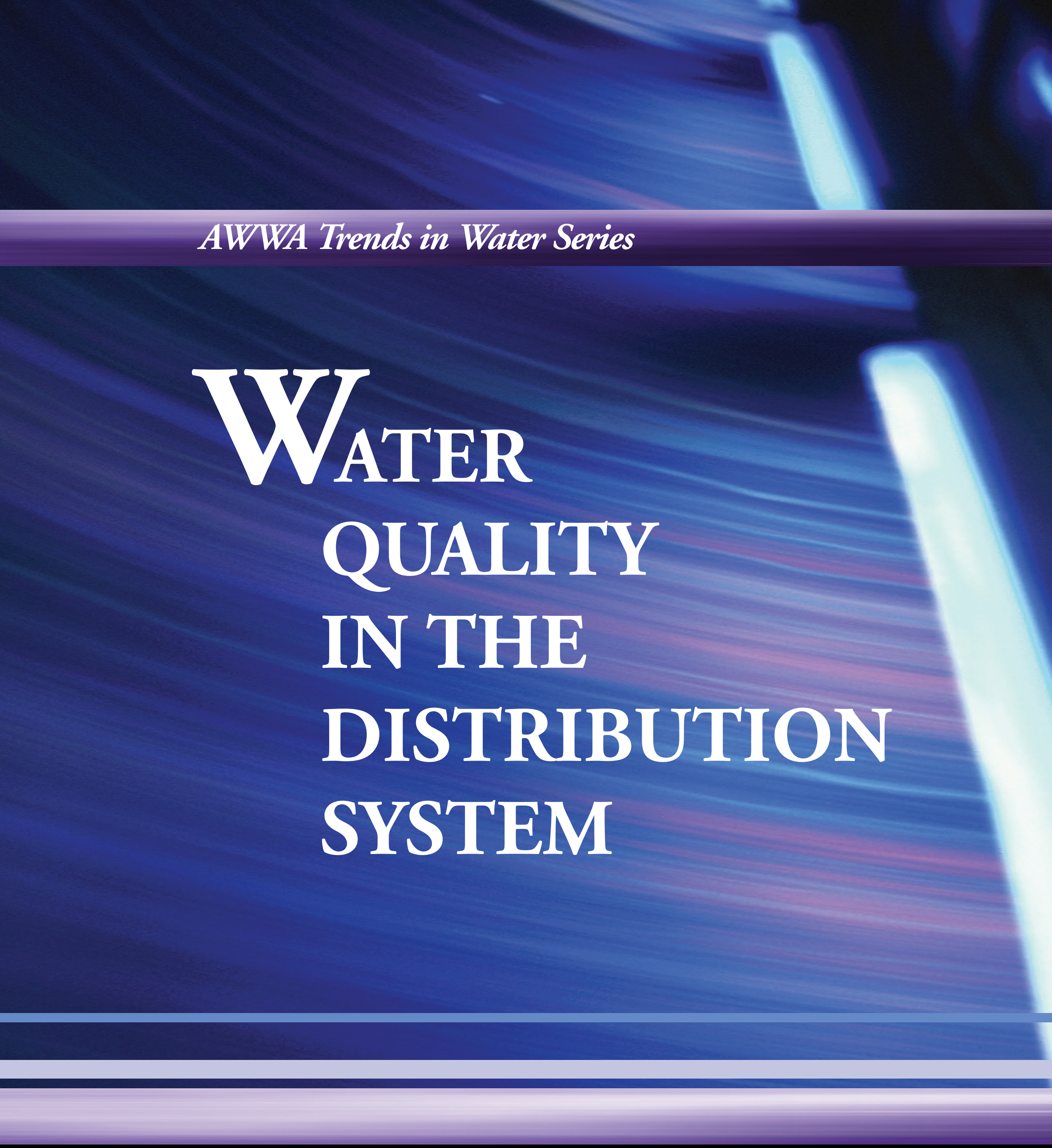

AWWA Trends in Water Series

WATER QUALITY IN THE DISTRIBUTION SYSTEM

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The Authoritative Resource on Safe Water^{SM}

Water Quality in the Distribution System

Water Quality in the Distribution System

English Language
First Edition

William C. Lauer, Technical Editor

American Water Works
Association

Science and Technology

AWWA unites the drinking water community by developing and distributing authoritative scientific and technological knowledge. Through its members, AWWA develops industry standards for products and processes that advance public health and safety. AWWA also provides quality improvement programs for water and wastewater utilities.

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Preface

The water distribution system has a direct impact on customer satisfaction. It is the final component in a multiple barrier approach utilities employ to deliver safe drinking water. Source water protection and management, treatment plant operation, and disinfection are the other barriers—each of which must be optimized to provide the maximum level of protection. Distribution system operators have traditionally focused their attention on a dependable supply at an adequate pressure. Customers and regulators are now demanding an increased emphasis on the water quality.

Several regulations focus directly on water quality in distribution systems. These include the Total Coliform Rule, the Lead and Copper Rule, and the Disinfectants/Disinfection By-products Rule. AWWA has also developed standards for distribution system operation and maintenance. Improved safety and customer satisfaction are the goals of all of these efforts.

This publication includes a selection of papers presented at AWWA events and published in AWWA periodicals. These papers were carefully selected from the hundreds available on this important topic. The articles included in this volume are taken primarily from the *Journal AWWA*, the AWWA Water Quality Technology Conference, the AWWA DSS Conference, and the AWWA Annual Conference (January 2001–January 2004).

The topic of maintaining water quality in distribution systems is diverse and complex. Therefore, this book is divided into nine major headings: Introduction, Microbiological Issues, Chemical and Physical Issues, Chloramine Conversion Issues, Corrosion Control, Rapid or Real-Time Monitoring, Operational Practices, Flushing to Maintain Water Quality, and Water Quality Computer Modeling (Computer-Aided Network Analysis). Readers may use these headings to help them find information on specific areas of interest without the need to read the entire volume. Many references are provided to assist those wishing to delve deeper into these areas.

The July 2001 issue of the *Journal AWWA* carried the theme Distribution Systems. The Introduction to this book is an article from that issue that described the results of an American Water Works Association Research Foundation report of five steps to prevent water quality problems. The article provides a comprehensive overview of the most common problems that lead to degraded water quality.

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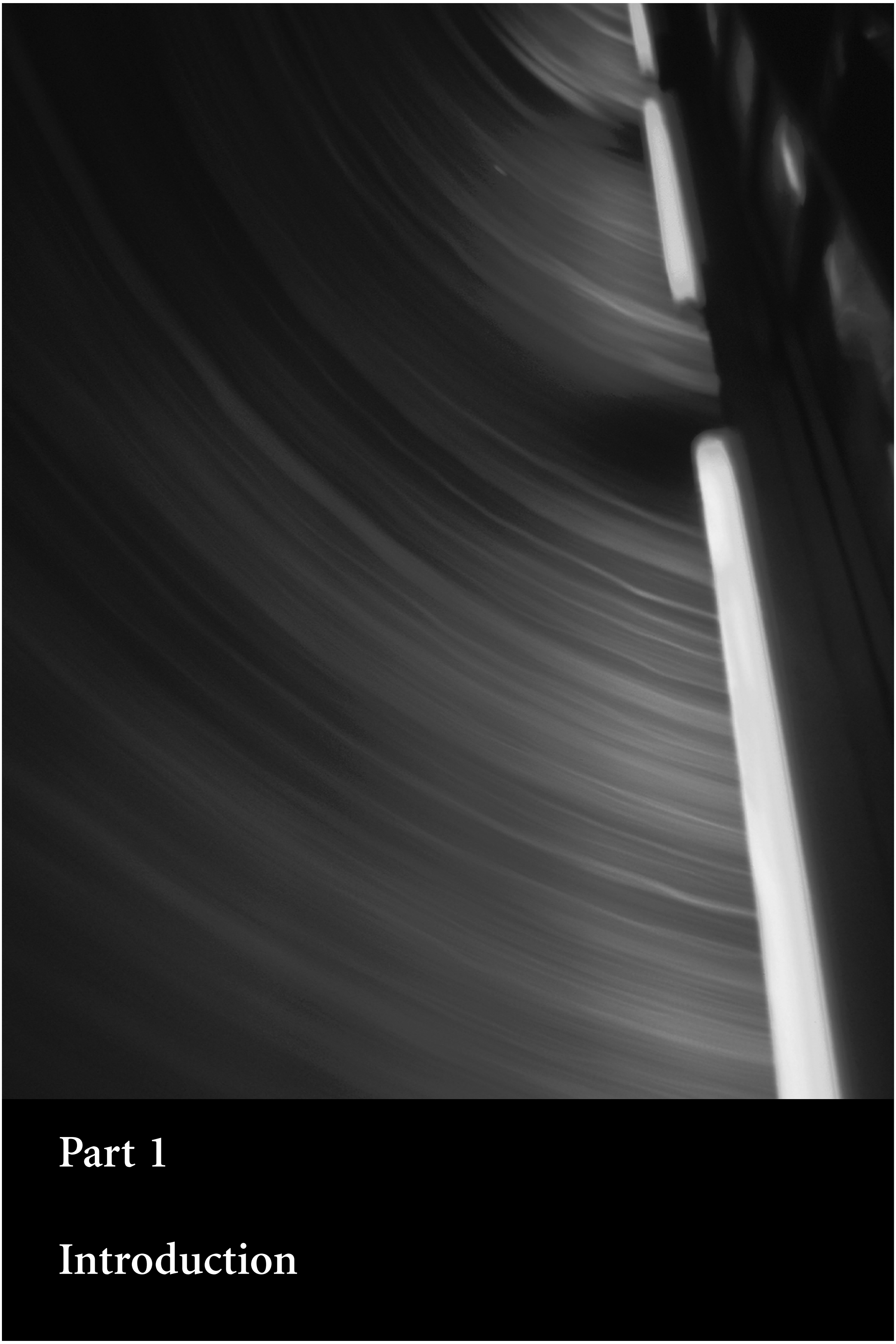

Part 1

Introduction

CHAPTER · 1

Practical Guidelines for Maintaining Distribution System Water Quality

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BACKGROUND

Much emphasis has been placed on the multiple-barrier approach for protecting public health as it relates to public water supply. Historically, most of the emphasis has been placed on source water protection, water treatment, and disinfection processes. The distribution system, including piping, storage, and appurtenances, is a protective "barrier" that needs to be operated and maintained to prevent contamination as water travels to the customer. It is an integral part of the multiple-barrier approach designed to protect water from source to tap.

Historical design procedures have tended to oversize pipelines and storage facilities, resulting in long detention times, loss of chlorine residual, taste and odor occurrences, and other water quality problems. From an operations standpoint, many storage facilities have been kept full in order to be better prepared for emergency conditions, resulting in long detention times and water quality degradation. Further, some materials that have been selected and installed in distribution systems have produced conditions suitable for increased growth potential of microorganisms—a situation that has regulatory implications. Unlined or exposed ferrous materials used in pipelines can corrode, causing "red" or rusty-colored water to occur. In conclusion, there needs to be a better balance between "hydraulics" or water supply needs and "water quality" considerations.

FIVE-STEP APPROACH RECOMMENDED FOR OPTIMIZING DISTRIBUTION SYSTEM WATER QUALITY

To optimize water quality through to the consumer's tap, the utility should consider following these five steps:

Step 1—Understand Your Distribution System and Define the Problems

This step is critical because it sets the tone and pace for optimizing water quality in the distribution system. It is accomplished by reviewing information such as water quality monitoring data, past reports on the system condition, as-built records, inspection and maintenance records, operating data, and the utility's management approach. The result should be an assets file that describes the physical facilities along with appurtenances and their intended uses, including piping, pumps, storage facilities, line and control valves, hydrants, blowoffs, and backflow prevention devices.

Water quality problems are classified

Water quality problems are classified as microbiological, chemical/physical, or aesthetic. Microbiological problems in distribution system water include bacteria proliferation/regrowth, nitrification, and waterborne disease. Chemical/physical water quality problems and parameters include disinfection by-product formation, lead and copper, temperature, corrosion, pH stability and scale formation, by-products of coatings and linings, maintenance of a disinfectant residual, and sediment. Aesthetic water quality problems include tastes and odors, color, and staining. It is important to determine whether the causes of water quality problems are related to interactions within the bulk water, from piping materials, from silt/sediment, or from direct chemical/microbial intrusion into the distribution system.

At this point, the utility will need to decide whether it has enough information to clearly define any problems. If not, then an expanded water quality monitoring program may be needed. Problems might be of a regulatory compliance nature, or they could be related to aesthetics and customer complaints.

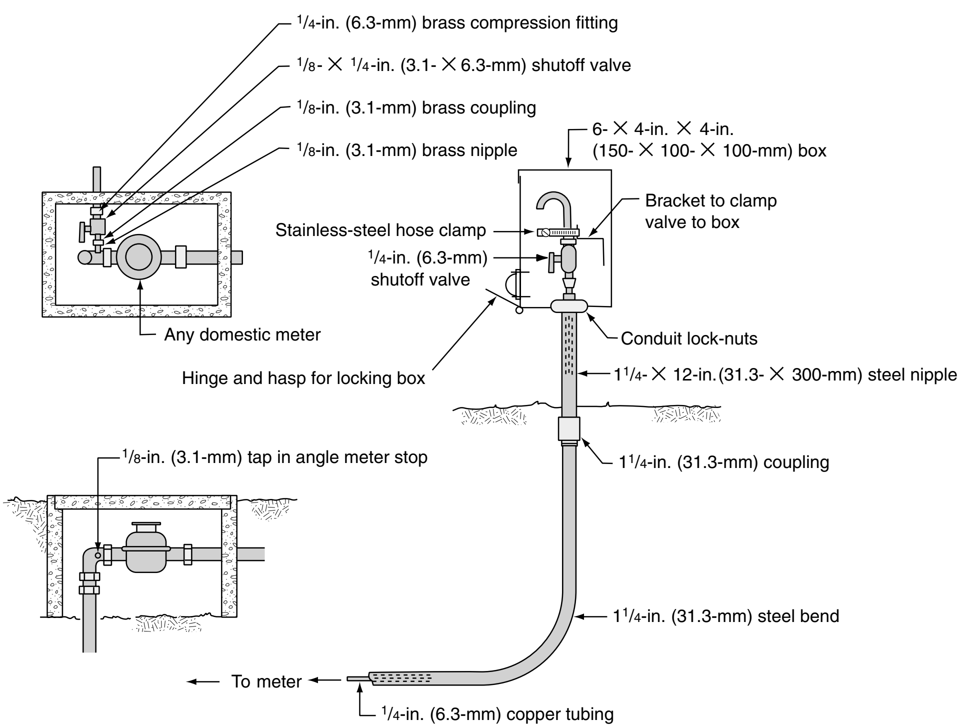
Distribution system engineering and design factors that can affect water quality include sizing of pipelines (left) and storage facilities (center) and the configuration of system components such as pumps (right).

Water quality monitoring is used to further characterize problems

A sampling plan should be developed to address issues such as selecting sampling sites, establishing test parameters and monitoring frequencies, establishing field monitoring protocols, and addressing laboratory considerations. To the extent possible, the location of sampling sites should be proportional to the populations served in all subregions of the system and should represent the range of water qualities accounting for both short and long water ages in the system. Ideally, sampling sites should be readily accessible, clean, and free of external contamination that could affect the sanitary quality of a sample collected for bacteriological examination.

The most commonly monitored test parameters for determining general distribution system water quality include coliform bacteria, heterotrophic plate count (HPC) bacteria, disinfectant residual, temperature, turbidity, pH, and color. Monitoring beyond the regulatory requirements is very useful to establish baselines for comparison of unusual values so those water quality excursions can be more readily detected. Special recognition should be given to HPC bacteria because they can be used as an indicator to help determine the microbiological condition of the distribution system. R2A medium (incubated at 20°C [68°F] for seven days) for HPC enumeration and m-T7 agar for total coliforms are sensitive techniques that can serve as an early warning for microbiological problems.

It is worth noting the importance of coliform monitoring and the use of dedicated sampling stations. Several utilities have documented the benefits of installing and using dedicated sampling stands to collect their



Box should be located near a stationary object, such as a power pole, for protection, or place sufficient concrete around riser belowground.

Source: Water Distribution System Operation and Maintenance, A Field Study Training Program. *US Environmental Protection Agency Office of Drinking Water and California Department of Health Services, Sanitary Engineering Branch. Hornet Foundation Inc., Sacramento, Calif. (1989, 2nd ed.).*

Figure 1 Components of a dedicated sampling station

bacteriological samples (Donner, 1987; Principe, 1998). Use of dedicated sampling stations improves access and has been associated with significant reductions in the number of positive samples, presumably caused by the sampling site and not the water. The components of a dedicated sampling station are shown in Figure 1. Special instances such as main break repairs and installation of new water mains require immediate attention, and proven monitoring procedures need to be followed to document the absence of contamination. Monitoring parameters for various types of emergencies are summarized in Table 1.

Table 1 Water quality test parameters that are useful in distribution system emergencies

Parameter	Application/Interpretation
Disinfectant residual	A sudden loss can indicate a disruption in treatment or a system breach (i.e., cross-connection or main break); rapid increases signal a treatment facility or booster station overfeed.
Turbidity and color	Sudden increases generally indicate a system disturbance including main breaks, corrosion, cross-connections, firefighting or hydrant opening, flushing, scheduled maintenance or repairs, valve failures, and treatment failures.
pH, conductivity, alkalinity	Changes (increase/decrease) often accompany a contamination event; a rapid change in pH may also indicate a chemical overfeed at the treatment facility.
Fluoride ion	This is useful in leak investigations in which fluoridation is practiced (i.e., differentiating between treated water and groundwater); a rapid increase with low pH may also indicate a chemical overfeed at the treatment facility.

Regulatory requirements must be understood

The US Environmental Protection Agency is increasingly focusing on the distribution system barrier. Proper distribution system operation and maintenance will be a must to meet future regulatory requirements. The most relevant current federal drinking water regulations are shown in Table 2. In addition to drinking water quality regulations, a myriad of other environmental regulations deal with such issues as the discharge of chlorinated waters to storm drains and surface waters.

Table 2 Water quality parameters and associated regulations for water distribution systems*

Parameter	Sample Location	Regulatory Limit	Rule	Comments
Disinfectant residual†	Entry point to distribution system	Minimum 0.2 mg/L on a continuous basis	SWTR	Only applies to systems using surface water supplies. In United States, *Legionella* is also regulated by a treatment technique.
Disinfectant residual	Distribution system	MRDL chlorine 4.0 mg/L; MRDL chloramine 4.0 mg/L, running annual average	D/DBPR Stage 1	Surface water systems serving > 10,000 people
Disinfectant residual or HPC bacteria count†	Throughout distribution system	Detectable level of disinfectant residual or HPC bacteria count of 500 or less colony-forming units/mL in 95% of samples collected each month for any two consecutive months	SWTR	Only applies to systems using surface water supplies
Total trihalomethanes	Throughout distribution system	80 μg/L, running annual average based on quarterly samples	D/DBPR Stage 1	Surface water systems serving > 10,000 people
Sum of five haloacetic acids	Throughout distribution system	60 μg/L, running annual average based on quarterly samples	D/DBPR Stage 1	Surface water systems serving > 10,000 people

Table continued next page.

Table 2 Water quality parameters and associated regulations for water distribution systems* *(continued)*

Parameter	Sample Location	Regulatory Limit	Rule	Comments
Total and fecal coliform bacteria	Throughout distribution system	5% samples positive for total coliform bacteria, repeat samples positive for fecal coliform or *Escherichia coli* bacteria	Total Coliform Rule	Number of samples determined by population served
Lead and copper	At customers' taps	Action levels: lead 0.015 mg/L at 90%, copper 1.3 mg/L at 90%	LCR	Number of samples determined by population served
pH	Representative points in distribution system	Minimum of 7.0	LCR	

*SWTR—Surface Water Treatment Rule, MRDL—maximum residual disinfectant level, D/DBPR—Disinfectants/Disinfection By-products Rule, HPC—heterotrophic plate count, LCR—Lead and Copper Rule

†Disinfectant residual may be regulated for some systems using groundwater supplies under the forthcoming Ground Water Rule.

Table 3 Prioritization of utility and customer water quality concerns

Utility Perspective	Customer Perspective
Microbial safety	Safe product that meets regulations
Disinfectant residual maintenance	Free of excess chlorine
Taste and odor removal/prevention	Free of tastes and odors
Corrosion control	Good appearance
Low disinfection by-product formation	Uniform water quality

Customer expectations are important

In order to prevent and/or solve water quality problems in the distribution system, the utility needs to understand customer expectations and attempt to balance these expectations with regulatory requirements and the utility's own internal goals. As shown in Table 3, customer expectations are not always aligned with utility goals. Although the customer and the utility both value the safety and health aspects of the water, the customer seems to place much more emphasis on the aesthetics of the water, including the taste, color, odor, appearance, and uniformity. For example, the utility strives to maintain a disinfectant residual throughout the distribution system, whereas the customer would prefer not to taste or smell any chlorine. In addition, the customer seems to equate health with aesthetic aspects of the water, sometimes thinking that a poor-tasting water may not be safe to drink.

Basic Water Quality Goals

- Treated water should be noncorrosive.
- Uncontrolled scaling should not occur.
- Natural organic matter should be minimized.
- Assimilable organic carbon should be minimized.
- After-floc (the continued formation or carryover of floc material into the distribution system) should not occur.
- Treated water particle counts/turbidity should be minimized.
- Iron, manganese, and hydrogen sulfide should be removed.
- Adequate disinfection should be provided.

Distribution System Water Quality Guidelines for United Water New Jersey*

Bacteriological Guidelines

1. There should be an absence of coliform bacteria.
2. Heterotrophic plate counts should be below 100 colony-forming units/mL.

Chemical Guidelines

1. Chlorine residuals—includes water in storage tanks, surface water system—total chlorine should be between 1.8 and 3.5 mg/L, groundwater system—free chlorine should be between 0.3 and 0.7 mg/L.
2. Iron level should be below 0.1 mg/L.
3. Manganese level should be below 0.02 mg/L.
4. Lead/copper level should be 100% below action levels.
5. There should be no objectionable taste and odor.

*Guidelines apply to all distribution system locations including dead ends.

Figure 2 Sample distribution system water quality guidelines

Step 2—Set Water Quality Goals and Establish Preliminary Performance Objectives

This step will involve establishing utility water quality goals that may go well beyond simply complying with regulatory requirements. For example, the utility may wish to establish a disinfectant residual goal of 0.4 mg/L at all active points in the distribution system, or it may wish to establish a threshold for HPC as an early indicator of microbiological degradation. An example of one utility's water quality guidelines for the distribution system is shown in Figure 2. Some utilities have established goals for the aesthetics of the water at the customer's tap in an attempt to reduce complaints and increase customer satisfaction.

In addition to water quality goals, the utility should establish specific performance standards to help meet those goals. Some suggestions are as follows:

- Provide source water treatment so that stable water is entering the distribution system.
- Maintain a positive pressure—for example, 20 psi (138 kPa)—at all times at ground level, and meet fire flow requirements.

- Minimize detention time in the distribution and storage system, i.e., establish a maximum time in days for key parts of the system.
- Maintain a disinfectant residual throughout the distribution system at some preset level, e.g., 0.2 mg/L.
- Keep the distribution system clean by following good maintenance and operating procedures, i.e., have a comprehensive flushing program.
- Implement and emphasize a cross-connection control program.

Step 3—Evaluate Alternatives and Select the Best Approach

This step uses the information from steps 1 and 2 to develop, evaluate, and select the preferred approach to optimize distribution system water quality. Depending on the type of water quality problem, the most appropriate solution may require changes in operations, maintenance, and management practices or additional monitoring, or may require an engineered solution at the source or in the distribution system. Often more than one solution will be necessary. Potential solutions to several water quality problems are presented in Tables 4–6.

Source water treatment improves water quality and reduces operations requirements

Adequate and appropriate source water treatment can greatly improve distribution system water quality. For example, removing silt and sediment and reducing organic carbon at the source can reduce the type and frequency of distribution system operational activities, such as rerouting water to minimize detention time and booster disinfection requirements. It is important to realize that the most aggressive distribution system operation and maintenance programs, at best, can only *maintain* water quality conditions provided at the point of entry to the distribution system. Regardless of the types of source water treatment used at a utility, treatment operations should be optimized so that water quality goals can be achieved (see box on page 10).

Three key operational parameters affect water quality

Water quality deterioration within the distribution system is most often a result of interactions between the bulk water and pipe materials and bulk water chemistry. Three primary operations procedures are used to

Table 4 Potential solutions to bacteria/regrowth problems within the distribution system

Category	Possible Solution
Monitoring	Identify source and magnitude of problem by determining heterotrophic plate count levels using R2A medium (incubated at 20°C [68°F] for seven days), and locations of coliform occurrence using m-T7 agar; measure disinfectant residual levels and/or chloramine ratios within the distribution system; monitor for turbidity, temperature, and nutrient levels.
Operations	Increase disinfectant residual, decrease detention time, increase turnover in storage facilities; disinfect problem area water mains; periodically switch to alternate disinfectant (i.e., from chloramines to free chlorine); use supervisory control and data acquisition system to control pumping and storage facility turnover for chlorine residual maintenance; verify that no cross-connections are occurring.
Maintenance	Conduct routine unidirectional or zone flushing to remove sediment, scour biofilm, and ultimately decrease chlorine demand; clean water mains and storage facilities; inspect and maintain storage facility vent screens.
Source water treatment	Provide corrosion control treatment to minimize oxidation of pipe materials; remove other nutrients such as iron, manganese, sulfide, and methane; provide adequate disinfection (i.e., ensure contact times are met); use an alternate disinfectant such as chloramines if disinfection by-product levels are a concern.
Engineering	Line or replace corroded water mains; loop dead ends to avoid stagnation; install side-by-side smaller-diameter pipes versus one large-diameter pipe in areas of new construction; reconfigure inlet/outlet of storage tanks; provide booster disinfection within the distribution system; select mortars and linings that do not stimulate microbial growth.
Management	Support public education regarding the benefits of flushing; compare costs of implementing required source water treatment versus relying on distribution system operation and maintenance techniques; ensure adequate workforce to carry out operations and maintenance activities.

Table 5 Approaches for minimizing DBP* formation

Category	Approach
Monitoring	Conduct nonregulatory monitoring throughout distribution system to identify trouble spots; evaluate DBP formation through storage facilities and within pressure zones; monitor for DBPs, total organic carbon, pH, alkalinity, disinfectant residual, and temperature.
Operations	Implement and maintain operations programs that reduce/minimize detention time within the distribution system; increase storage facility turnover; estimate hydraulic residence time throughout system using appropriate, calibrated, hydraulic model.
Maintenance	Clean pipelines and storage facilities to remove sediment (DBP precursors); conduct routine flushing to move water through the system but also to remove sediments.
Source water treatment	Reduce organics level through biological filtration, ultrafiltration or nanofiltration, granular activated carbon filtration, enhanced coagulation; convert primary disinfectant from free chlorine to ozone or chlorine dioxide; convert secondary disinfectant from free chlorine to chloramines.
Engineering	Conduct secondary disinfection at lower, consistent levels rather than adding all disinfectant at the wellhead or treatment plant effluent (while maintaining Surface Water Treatment Rule contact time requirements); reconfigure storage tanks to improve circulation; eliminate dead ends through looping to increase water movement.
Management	Develop standard operating procedures for moving water; support public education regarding the benefits of flushing; compare costs of implementing required source water treatment versus relying on distribution system operation and maintenance techniques; ensure adequate workforce to carry out operations and maintenance activities.

* DBPs—disinfection by-products

Table 6 Approaches for minimizing distribution system tastes and odors

Category	Approach
Monitoring	Use customer complaint data to identify areas of distribution system with taste and odor problems; conduct monitoring in areas with frequent taste and odor complaints to identify causes; conduct source water monitoring to determine hydrogen sulfide levels, microbial conditions, concentrations of iron/manganese.
Operations	Minimize detention time in pipelines and storage facilities.
Maintenance	Conduct routine flushing in areas with excessive detention time, in areas with increased taste and odor complaints; review records showing piping types, age, location, maintenance records.
Source water treatment	Provide treatment such as granular activated carbon filtration or ozonation to remove tastes and odors associated with source water; ensure adequate disinfection; consider switching to chloramines if chlorinous tastes and odors are chronic.
Engineering	Consider booster chlorination at lower dosages to maintain chlorine residuals; consider taste and odor potential when materials are selected for the distribution system; cover storage facilities to limit algal growth; ensure proper applications and curing of coatings and linings.
Management	Provide information to the public regarding causes of tastes and odors; ensure that causes are identified and steps are taken to reduce episodes; train personnel who are in contact with customers to respond to inquiries and to explain actions utility is taking to address issues.

maintain water quality: (1) minimizing bulk water detention time, (2) maintaining positive pressure, and (3) purposefully controlling the direction and velocity of the bulk water.

The key elements associated with each operational goal are summarized in Figure 3.

Bulk water detention time should be minimized because both the bulk water decay characteristics and the interactions between the pipe walls and the bulk water result in a deterioration of water quality. This is evidenced by problems such as decay of the secondary disinfectant

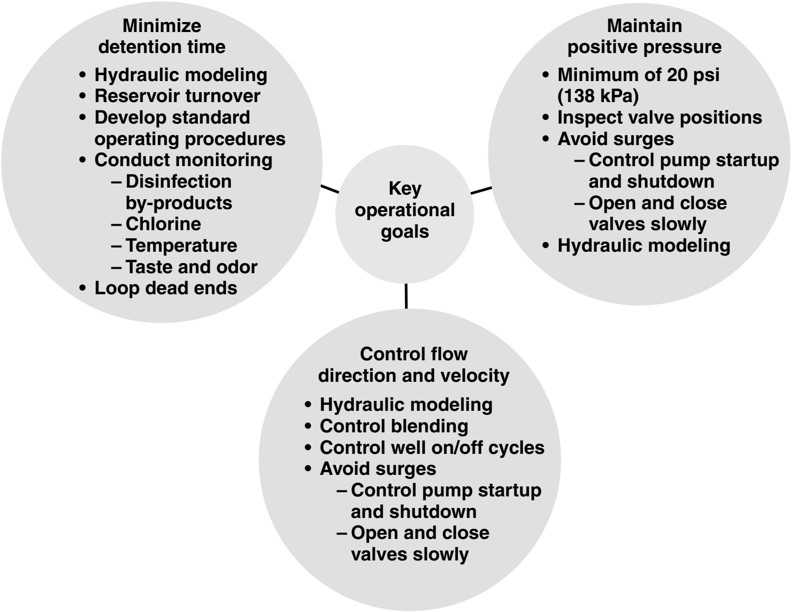


Figure 3 Key elements associated with distribution system operation goals

residual, increases in disinfection by-product (DBP) levels, and possibly increased taste and odor. Water quality models can be used to predict levels of residual chlorine and possibly DBPs. In addition, reactions between the water and the metallic pipe surfaces and the scales will result in higher levels of metallic by-products such as iron and copper as the exposure times increase. The age of water stored in finished water storage facilities contributes to the overall water age in the distribution system. As a starting point, it is recommended that stored water be completely turned over every three to five days at a minimum (Kirmeyer et al., 1999). Of course, this is site-specific and should be tailored to each utility.

A positive pressure should be maintained throughout the distribution system to minimize the potential for back siphonage or backflow of contaminants to occur. Various codes of good practice and manuals suggest 20 psi (138 kPa) as a minimum pressure to maintain at ground level or at all points throughout the distribution system under extreme operating conditions, such as high demand and fire flow circumstances (AWWA, 1996; Health Education Services, 1997; USEPA & CDHS, 1989).

Computerized hydraulic models can be used to help predict and maintain water pressure. Cross-connection control programs need to be emphasized by water utilities and health agencies. These programs need to include program development, regulatory authority, inspection and testing, recordkeeping, and education.

Utilities should attempt to minimize rapid and/or extreme fluctuations in flow velocities and should minimize the frequency of flow reversals. Activities that may affect flow velocities include rapidly opening or closing a valve, a power loss, and hydrant flushing. Changes in flow velocity can scour sediments, tubercles, and deposits, releasing particles into the water and causing water quality deterioration. In addition, rapid changes in flow velocity create pressure surges where very high pressures are followed by low and negative pressures. When the pressure surrounding the water main exceeds the internal pressure, any water present in the trench may flow into the main through leakage points, submerged air valves, cross-connections, faulty seals, or joints.

Maintenance alternatives include flushing, cleaning, and repairs

Water system flushing is an important tool for helping to keep the water system clean and free of sediment, to remove stagnant water, and to remove an unwanted contaminant that may have inadvertently entered the system. It is important for the utility to establish specific goals and objectives for the flushing program. Then the program and field practices can be developed to accomplish the goals in a cost-effective manner. Major issues will be the type of flushing program, i.e., spot flushing or a more systemwide approach; unidirectional flushing versus conventional flushing; the frequency of flushing; the target velocity needed to accomplish the objectives; monitoring; and recording the information gathered.

A variety of cleaning techniques may be used for pipelines. These techniques include mechanical scraping (Scanga & Guttman, 1992; AWWA, 1987), pigging (Jones, 1987), swabbing (Jones, 1987), chemical cleaning (Plishka & Shenkiryk, 1996), and flow jetting. Each technique has its benefits and drawbacks and should be tailored to the specific site. When cleaning is used, it may or may not be followed by relining of the pipeline. If the pipe interior is not to be lined, special consideration should be given to the type of material and its condition so that corrosion of the newly exposed surface does not rapidly occur. Finished water storage facilities need to be cleaned periodically to remove silt, sediment, biofilms, and other accumulated material. These facilities may be cleaned by a variety of methods while they are full of water or are empty. The traditional method of draining and cleaning enables the bottom and the lower portions of the reservoir walls to be cleaned thoroughly but requires the facility to be removed from service for long periods of time. Newer techniques that use divers or remotely operated vehicles may not facilitate cleaning quite as thoroughly, but they have the significant benefit of reducing the time that the facility is out of service. Additional information on maintaining storage facilities can be found in a AWWA Research Foundation guidance manual (Kirmeyer et al., 1999).

Utility maintenance activities also include emergency pipe repairs with sanitary precautions in place. Sanitary practices include keeping contaminated water out of the trench and pipe as much as feasible, flushing the line in the vicinity of the break, applying disinfectant to the components that were potentially contaminated, and conducting bacteriological testing of the water to confirm the absence of contamination. Sanitary practices are also necessary in the construction and release of new water mains. Disinfection of new mains should follow AWWA Standard C651 Disinfecting Water Mains or the applicable state regulations.

Engineering and design alternatives reduce water age

The first chance that water system operators have to consider water quality is during system design. System configuration or reconfigurations can be evaluated in terms of water quality, specifically evaluating water age as the water travels through the system. Computerized hydraulic and water quality models can be used to estimate water age and chlorine levels. Pressure zones can be planned or reconfigured to reduce water age and maintain water quality. If the utility is experiencing problems with low chlorine residuals, it may want to consider installing a booster disinfection station. Models can also be used to help site and size finished water storage facilities. Dead-end pipelines should be avoided, or special precautions should be taken to preclude excessive water age.

The design of pipelines can significantly affect water quality. The sizing of lines needs to strike a balance between water quality and the cost for future expansion. The materials and the linings used to coat pipelines should be specified to be NSF International– or Underwriters Laboratories–approved and should be compatible with the utility's water quality. Depending on the location of the pipeline and the potential contaminants in the soil, certain types of pipe should not be used because volatile organic chemicals may permeate through the material.

Deteriorated pipelines, especially unlined cast-iron lines, can create water quality problems such as rusty or red water, can reduce the secondary disinfectant levels, and can support biofilm growth, which can have regulatory implications. The utility is often faced with the dilemma of deciding whether to rehabilitate the pipe or replace it. The costs for each technique, the structural condition of the pipe, the disruption to customers, plus other factors, must be considered in making the decision. These types of programs are costly and should be carefully planned as part of a capital improvement planning process.

Finished water storage facilities should also be designed to facilitate maintenance of water quality. In most cases, chlorine contact time is not an issue in distribution storage; thus, finished water storage facilities should normally be designed to enhance mixing rather than to promote plug flow conditions. There are several ways to promote mixing, including a momentum-based approach in which the energy of the water entering the reservoir is used or an approach in which additional energy is input to the water by mechanical or hydraulic means. Models can also be used to predict the flow and mixing characteristics in finished water storage facilities.

Management alternatives include strategic planning, regulatory compliance, and customer relations

Management's role involves developing, applying, and continuously improving an array of management practices including funding, customer relations, regulatory compliance, investing in new technologies, and strategic planning. Managers need to ensure that there is a customer-driven approach for managing distribution systems and that good communication is maintained with the customer about the condition of and need for repair of hidden infrastructure. It is important for field crews to be trained in customer relations because they are the representatives whom the customer normally sees and interacts with.

Managers need to ensure that the utility invests in new technology to become more efficient. Examples of this include supervisory control and data acquisition systems and laboratory information management systems, which both offer state-of-the-art technology to better respond to water quality changes and needs. In addition to regulatory compliance, the progressive utility will want to establish internal goals to meet or surpass the regulatory standards. This will provide a margin of safety when problems do occur.

Water quality monitoring, performed at the Greeley–Bellvue Water Treatment Plant in Bellvue, Colo., is a critical part of the first step to optimizing distribution system water quality.

Step 4—Implement Good Management Practices and Monitor Effectiveness

This step puts the recommended plan from step 3 into action. It will require coordination and cooperation of various utility staff and possibly outside specialists. Initially, diagnostic monitoring of effectiveness may be required, followed by more routine procedures. Adjustments to good management practices may be needed to improve results.

Step 5—Finalize Performance Standards and Develop Standard Operating Procedures

This step will require the multiple operating units within the utility to join together to develop standard operating procedures (SOPs) for the facilities in question. The preliminary performance standards proposed in step 2 should be revisited and changed if needed to reflect lessons learned during implementation. A general SOP addressing all distribution facilities may be developed, and an SOP for monitoring, inspection, maintenance, and operation for groups of or individual facilities may be in order.

SOPs can serve as a training guide and can help pass down knowledge from experienced staff to those who are assuming increased responsibility. Comprehensive water quality monitoring programs need to be well documented, including sample site locations, standard routes, frequency of collection, parameters, laboratory procedures, quality control, field analysis techniques, and so forth. For purposes of water quality

Each water utility should develop a sampling plan to select sampling sites, establish test parameters and monitoring frequencies, and address laboratory considerations.

protection, the utility should have routine inspection procedures for the condition of pipelines and storage facilities. Adequate recordkeeping practices are a must to provide a basis for operations and maintenance to enhance water quality. Finally, emergency response procedures need to be documented, and they should be practiced in both the office and the field. It is especially important to test the communications systems to improve emergency response capabilities.

CONCLUSIONS

The recommended stepwise approach to optimizing distribution system water quality is a practical method for utilities to apply. In step 1, water quality issues are defined by developing an assets file and reviewing system data. Water quality problems are classified and data needs are identified. Additional water quality monitoring is often necessary. Step 2 involves establishing water quality goals and setting preliminary performance objectives. In step 3, multifaceted solutions are developed, including treatment, operations, maintenance, engineering design, and management elements. Source water treatment should be optimized to remove iron, manganese, and hydrogen sulfide and to minimize natural organic matter, assimilable organic carbon, and turbidity levels in the finished water. Treated water should be noncorrosive and adequately disinfected. Operations solutions may include minimizing bulk water detention time, maintaining positive pressure, and purposefully controlling the direction and velocity of the bulk water. Maintenance solutions include cleaning, painting, and making repairs. Engineering and design

Following good maintenance and operating procedures—including having a comprehensive flushing program—can help keep the distribution system clean.

alternatives affecting water quality include system configuration, sizing of pipelines and storage facilities, and selection of materials. Management alternatives include strategic planning, regulatory compliance, and customer relations. Step 4 puts the recommended plan from step 3 into action and requires good management and monitoring to ensure its effectiveness. In step 5, performance standards are finalized and SOPs are developed.

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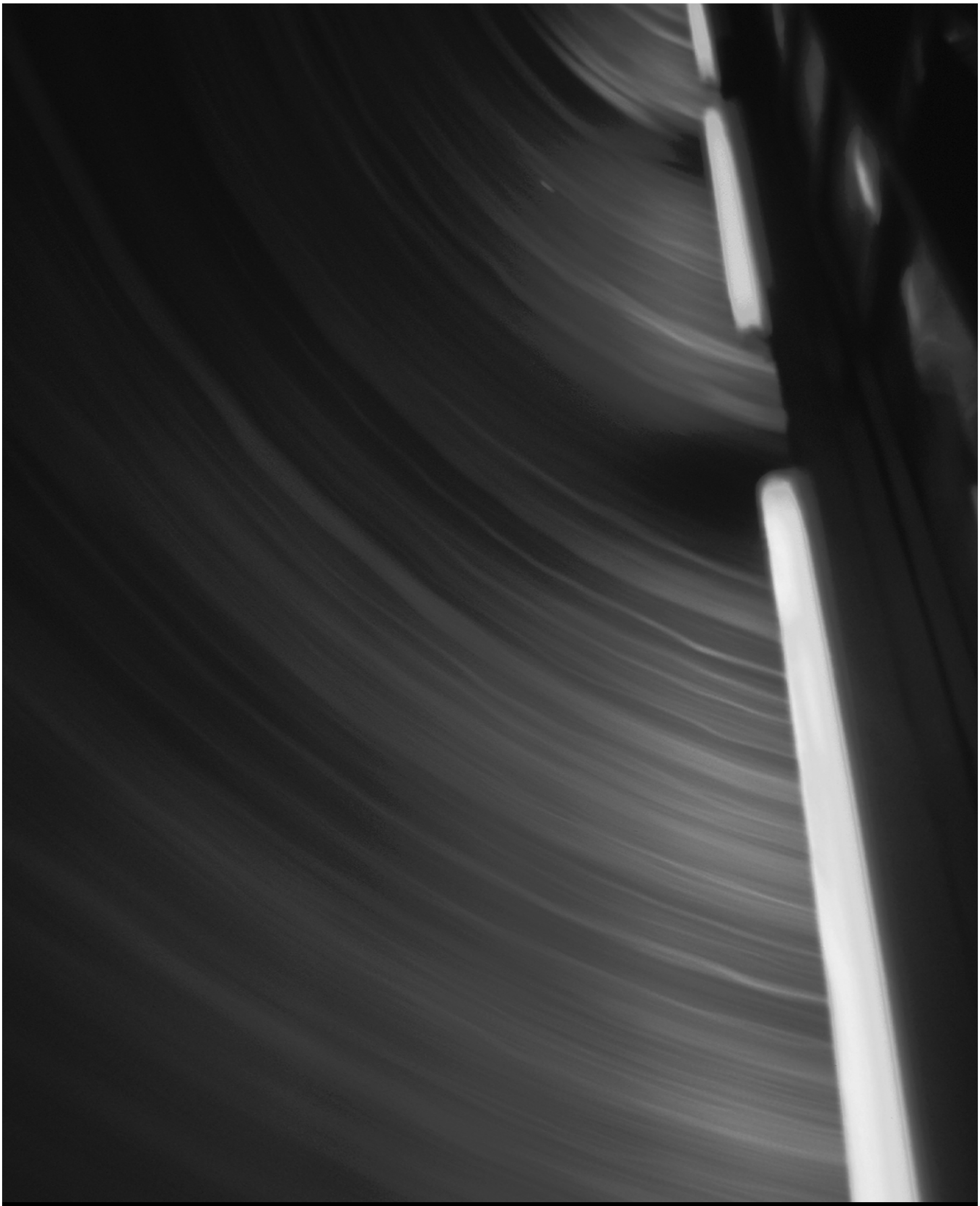

Part 2

Microbiological Issues

CHAPTER · 2

Explaining the Occurrence of Coliforms in Distribution Systems

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BACKGROUND

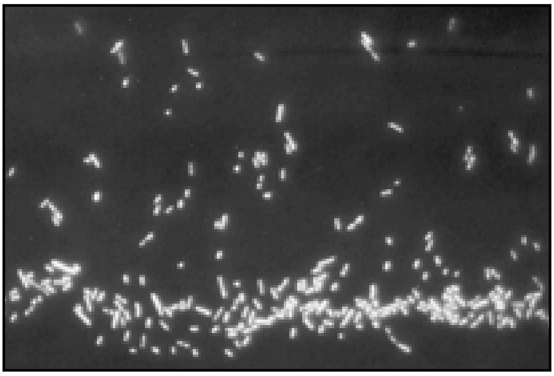
According to the Total Coliform Rule, coliform bacteria constitute the main indicator used to detect microbial contamination in distribution systems. A major goal for water utilities is to prevent and control coliform occurrences and noncompliance (with respect to regulations). However, the applied solutions are sometimes successful only to a limited extent because of the variety of factors that may give rise to these occurrences. It is therefore important to identify those factors—from the structure and operation of the distribution system to the quality of the distributed water itself—that can influence the occurrence of coliform bacteria in a distribution system. The use of the identified factors makes the modeling of coliform occurrences attractive, and a number of approaches for doing so have recently been proposed. This chapter provides a review that includes the mechanisms of how coliform bacteria are introduced into treated and distributed drinking waters, the major factors controlling the survival and regrowth of coliforms once introduced into the system, and the modeling efforts carried out to explain or predict their occurrence.

The coliform group of bacteria is used worldwide as an indicator of the microbiological quality of drinking water (WHO, 1993). The group consists mainly of several genera of bacteria belonging to the Enterobacteriaceae family, mostly *Escherichia*, *Klebsiella*, *Enterobacter*, *Citrobacter*, and *Serratia*. The detection of coliforms in tap water is supposed to indicate to utilities and health authorities that a breach has occurred in treatment or in the distribution system, allowing microbial (fecal) contamination of water to occur and thereby rendering it inappropriate for drinking.

In practice, coliforms are found quite often in tap water. However, the magnitude of the health risk associated with their presence has been somewhat mitigated—a tolerance of 5% for water samples testing positive for nonfecal coliforms on a monthly basis is allowed in the United States (USEPA, 1989). This was done to take into account the fact that several of these microorganisms occur naturally in the aquatic environment, originating from soil and vegetation (Kreig, 1984) or industrial activities (Archibald, 2000). Because many coliforms are able to survive and regrow in a distribution system (LeChevallier, 1990), their presence does not necessarily indicate recent contamination. It is also worth mentioning that bottled water has been shown to present coliform occurrence rates that are similar to those in tap water samples (Warburton et al., 1998).

The validity of using coliforms as indicators of the microbiological quality of drinking water may be questioned because coliforms are sometimes detected when there is no evidence of contamination and they are sometimes not detected when pathogenic organisms are found in the water. Craun et al. (1997) reported coliform detection during most (64%) of the waterborne illness outbreaks caused by bacteria, viruses, and unidentified agents from 1983 to 1992 in the United States. However, coliforms were detected in only a few (35%) outbreaks caused by protozoa. Similar results have also been reported in the United Kingdom (Furtado et al., 1998). Other studies (Payment et al., 1997; Zmirou et al., 1995; Payment et al., 1991) suggest that there is a low but significant risk of gastrointestinal illness from the consumption of water meeting the current bacteriological regulations. To improve the link between fecal contamination and regulations, *Escherichia coli* has been proposed as a better indicator than total coliforms since simple analytical techniques have recently been made available (Edberg et al., 2000).

However, until the Total Coliform Rule (TCR) is revised, water utilities will continue to use total coliform bacteria to assess the microbiological quality of their drinking water to comply with US regulations (Pontius, 2000). Consequently, efforts should be continued to prevent and control total coliform occurrences and noncompliance (with respect to regulations) and their possible association with contamination of distribution system waters. It is thus important to identify the factors—from the structure and operation of the distribution system to the quality of the distributed water itself—that can influence the occurrence of coliform bacteria in the distribution system (Craun &



Suspension of coliform bacteria (*Enterobacter cloacae*) is observed by epifluorescence microscopy after staining with DAPI.

Calderon, 1999). With this in mind, this chapter provides a review including the mechanisms of how coliform bacteria are introduced into treated and distributed drinking waters, the major factors controlling the survival and regrowth of coliforms once introduced into the system, and the modeling efforts made to explain or predict their occurrence.

MECHANISMS OF COLIFORM INTRODUCTION

Two primary mechanisms are responsible for the introduction of coliform bacteria into a distribution system. Coliforms may break through into treated water as a result of inadequate treatment or be introduced as a result of intrusion into the distribution system downstream of the treatment plant (Craun & Calderon, 2001). These two pathways are usually very difficult to distinguish, because only a very small fraction of the total amount of treated water is tested for coliforms, and intrusion into the distribution system may be very localized in time and space. In some cases, epidemiological studies have helped in identifying that leakage and cross-connections, associated with low pressures, were responsible for tap water contamination (Semenza et al., 1998). In other cases, the detailed identification of the microorganisms involved has helped in identifying their origin (O'Neill & Parry, 1997; Edberg et al., 1994). But, in most cases, the origin of coliforms found in distribution systems remains unclear. Recently, one study (Kirmeyer et al., 1999) proposed a prioritization of entry routes into the distribution system (Table 1). The study concluded that as in treatment breakthrough,

Table 1 Prioritization of pathogen entry routes into distribution systems*

Entry Routes Priority	Ranking
Water treatment breakthrough; transitory contamination; cross-connection; water main repair or breakage	High
Uncovered storage facilities	Medium
New main installation; covered storage facilities; deliberate contamination	Low

*Modified from Kirmeyer et al., 1999

intrusion from transitory low-pressure events, cross-connections, and pipe repairs constituted a high risk of contamination.

Treatment Breakthrough

Coliform presence in treated water is seldom reported, which is logical because the treatment is usually designed to inactivate or at least remove bacterial contamination, and its efficiency is evaluated using coliform-indicator bacteria. Nevertheless, this absence of coliforms in treated water may only be apparent because of the very small fraction of treatment effluents monitored, making the detection of a time-limited breakthrough illusory (the fraction of the volume sampled for coliform analysis usually ranges from $1/10^6$ to $1/10^9$). Low percentages of treated water samples testing positive for coliform bacteria (<0.2%) have been reported in treated water from two Canadian treatment plants, one of which uses granular activated carbon (GAC) filtration (Morissette et al., 1999). It is not known whether these coliforms pass through the filters or grow in them. Colonization of GAC filters with coliforms was demonstrated by Camper et al. (1985) and was also suspected by Montiel and Welté (1999), who systematically recovered thermotolerant coliforms from the effluent of GAC filters during periods of high summer temperature (> 80% of the samples tested positive).

Coliforms may also remain undetected as a result of the cultivation techniques used for coliform detection—some or all of the coliforms may be injured (and not killed) by disinfection and thus unable to grow on traditional agar media. Various modified cultivation techniques have been proposed to permit the enumeration of injured coliforms (McFeters, 1990; McFeters et al., 1986; LeChevallier & McFeters, 1985).

Using modified cultivation techniques, McFeters et al. (1986) found injured coliforms in the effluent of three US facilities, and Craddock and Castle (1997) detected injured coliforms in 10.4% of 173 treated water samples from four treatment plants in the United Kingdom, while routine bacteriological tests indicated that only 0.6% of the samples were coliform-positive. The injured coliforms were found in the effluent of the three tested plants treating surface water, while they were undetected in treated borehole water, and positive samples were mainly associated with backwashing of the sand filters.

Release of particulate matter in treated water may facilitate the introduction of coliforms into the distribution system and may play an important role in treatment breakthrough. Examination of the particles released from GAC filter beds (Camper et al., 1986) showed that up to 17% of the samples contained carbon particles colonized with coliform bacteria. It has, in fact, been shown that cellular aggregation and attachment of bacteria to particulate matter result in the protection of microorganisms from disinfection (Gauthier et al., 1999a; Morin et al., 1999; Stringfellow et al., 1993; Berman et al., 1988; Herson et al., 1987; Ridgway & Olson, 1982).

Coliforms may also be associated with other types of particles such as invertebrates. Although most of the invertebrates that enter a water treatment plant are usually trapped and removed, organisms such as nematodes, rotifers, and protozoa have been observed in treatment plant effluent by many authors (Gauthier et al., 1997; Schreiber et al., 1997; Van Lieverloo, 1997; Brazos & O'Connor, 1996).

The presence of these organisms in treatment plant effluent may contribute to the introduction of coliforms into the distribution system. Schoenen and Hoyer (2000) have related the detection of total coliform in the treated water of a German plant to the presence of the larvae of gnats on filters and enumerated as many as 2×10^7 nonfecal coliform colony-forming units (cfu)/gnat larva. Moreover, laboratory studies have shown that organisms such as protozoa (amoebae and ciliates), nematodes, and amphipods can ingest bacteria such as *Escherichia coli* or *Enterobacter cloacae* and protect them from disinfection (Ding et al., 1995; King et al., 1988; Levy et al., 1986).

Another study (Lupi et al., 1995) conducted analyses on the intestinal contents of nematodes isolated from treated surface water and detected the presence of Enterobacteriaceae of environmental origin (up to 13 cfu/nematode), the majority of which were *Serratia*. Particulate matter and invertebrates might thus protect some coliforms from treatment plant disinfection and transport them into the distribution system.

In most cases, the presence of coliforms in treated water results from inadequate treatment settings or performance with respect to raw water quality. High bacterial loads in the source water are often a result of rainfall (Geldreich, 1996) and have been associated with higher coliform occurrences in the distribution system (LeChevallier et al., 1996b; LeChevallier et al., 1991). Several hypotheses may explain such occurrences, including less efficient treatment in such conditions and change in organic matter causing coliform regrowth downstream in the distribution system. The latter hypothesis is supported by the lag between rainfalls and distribution system coliform occurrences (LeChevallier et al., 1991).

Contamination of groundwater following heavy rains combined with inadequate chlorine disinfection resulted in the contamination of the Walkerton distribution system (Canada) with *E. coli* O157:H7. The resulting major outbreak affected 1,346 people and killed at least 6 people (Health-Canada, 2000). Contamination of groundwater also resulted in recent *E. coli* outbreaks at a New York fair (Yarze & Chase, 2000), in Ontario (Jackson et al., 1998), in Europe (Chalmers et al., 2000), and in viral outbreaks that were initially detected as a result of analysis for fecal coliform (Häfliger et al., 2000; Kukkula et al., 1997). The protection of groundwater catchment areas is crucial to avoid contamination. For example, one study (Raina et al., 1999) observed significant association between the presence of *E. coli* in domestic well water and gastrointestinal illness. In this case, the relationship depended on the distance of the well from the house's septic tank. In contrast, no recent outbreak has been associated with the failure of surface water treatment plants to inactivate coliform organisms.

Therefore, the potential exists for coliforms to penetrate into distribution systems. Coliforms may be present even if they are not detected as a result of insufficient sampling or difficulties in measuring them, either because of their disinfectant-induced injured state or because of their association with (bio)particles.

Intrusion

A high potential risk of introduction of coliform bacteria and enteric viruses into the distribution system exists when transient negative pressure occurs in pipelines (LeChevallier et al., 1999). The study conducted by these authors indicates that fecal indicators and culturable enteric viruses can be detected outside distribution pipelines. Consequently, in a case in which there are pipe leaks, there exists a potential portal of entry of microorganisms into the distribution system during negative pressure events originating from pump starting or stopping, transmission main breaks, rapid opening or closing of valves, loss of power, hydrant flushing, and other conditions, especially on elevated sites (Kirmeyer et al., 1999). Illustrating this path of intrusion, McMath and Casey (2000) measured subatmospheric pressures at one distribution system point downstream of the pumping station of a UK system during surges. They observed that this point (an air valve chamber) was flooded with dirty water during wet weather, thus explaining the high coliform failure rate in the corresponding distribution system area.

Contamination of the distribution system may also occur when a water main breaks or is repaired. Two major distribution line breaks were the suspected sources of contamination in the Cabool, Mo., outbreak of *E. coli* O157:H7, which caused 243 known cases of diarrhea and four deaths (Geldreich et al., 1992). Moreover, contamination may also occur if cleaning and disinfection procedures following repair are not adequately executed. LeChevallier (1999) noted that the flushing velocity of a repaired pipe is not always sufficient (often too low) to totally remove contamination. Pizzi (1996) also noted that a negligent worker with only a small amount of dog feces on the sole of his boot could contaminate many kilometers of pipes if disinfection following repair was inadequate. Contaminated sediment introduction during pipe repairs was also the main suspected cause of recurrent coliform occurrence in a dead-end area of a Canadian distribution system studied by Gauthier et al. (1999b). Another study (Haas et al., 1999) reported that for 16% of the utilities surveyed, 1% of the first samples taken following new main disinfection showed a positive result for total coliform. These authors supported—and completed—the current AWWA standards for such disinfection procedures.

Cross-connections, defined as connections between a potable drinking water supply and a nonpotable, undesirable, or contaminated source, may also pose a threat to distribution system integrity (Herrick, 1997). Coliform bacteria and contaminated water may be introduced into distribution pipes from the backflow of water through the cross-connections because of a differential in pressure between the connected systems, especially during low-pressure events in the distribution system. Lahti and Hiisvirta (1995) reported two outbreaks in Finland caused by cross-connections involving sewage and seawater, both showing evidence of the presence of *E. coli*.

Numerous backflow incidents that have occurred throughout the United States are listed on the Web site of the American Backflow Prevention Association (1995), demonstrating that almost any kind of liquid can penetrate a distribution system through backflow. Garden hoses are among the most common causes of cross-connections (Herrick, 1997), and fire sprinkler systems may also be problematic (Dubay, 1997). Nevertheless, a detailed investigation of the water quality in 84 wet-pipe fire sprinkler systems (Duranceau et al., 1998) has shown that total coliforms were mostly absent from those systems and that the main risk of microbial contamination of the distribution system through backflow remains directly linked to the intrusion of sewage or raw water. Such an intrusion could eventually result in the colonization of the distribution system with coliforms, but this has not been confirmed experimentally—two separate studies (McMath et al., 1999; Sibille et al., 1997) were not able to recover fecal contamination indicators following the passage of a slug-dose of sewage in an experimental distribution system.

Uncovered finished water reservoirs located on distribution systems represent another potential source of fecal contamination of potable water supplies from birds and other animals. LeChevallier et al. (1996b) have confirmed that the presence of open finished water reservoirs is one of the factors responsible for increasing the number of coliform occurrences. Water in closed storage tanks may also be contaminated if maintenance is inadequate. An outbreak of *Salmonella typhimurium* in Missouri probably resulted from bird contamination in a municipal water storage tank (Clark et al., 1996). The tank had an inappropriate roof vent and an uncovered hatch, which allowed free access to wild birds. Feathers were also discovered in the tank. For concrete tanks, contaminated water ingress through a tank's roof may also be detected by coliform presence in the water, indicating that remedial action (roof repair) needs to be taken (O'Neill & Parry, 1997).

Identification of the Source of Entry

For many utilities, the origin of coliforms found in distribution systems remains unclear, and multiple solutions are applied to eliminate them (Kirmeyer et al., 2000). For example, the Washington, D.C., water supply system experienced a series of microbial violations under the TCR between September 1993 and July 1996 (Clark et al., 1999). The passage of coliform organisms into the distribution system resulting from treatment barrier breakthrough was evidenced by the measurement of total coliforms in treated water samples, but a sanitary survey of the storage facilities and of the distribution system also indicated numerous

Unidirectional flushing may remove deposit-associated coliforms from distribution systems.

operational and maintenance deficiencies in the system, which could favor the intrusion of coliform bacteria into the distribution system from an external source (Clark et al., 1999).

The City of Seattle also experienced total coliform occurrences in 1993 and attributed them to a number of factors (Table 2). Short-term actions consisted of the adjustment of disinfectant residuals and the implementation of unidirectional flushing, which was very efficient in controlling coliform occurrences (Oliver & Pimentel, 1998; Oliver & Harbour, 1995). The number of positive samples for coliforms at the Metropolitan Water District of Southern California was reduced by improving sampling point protection and cleaning, thereby preventing airborne contamination (Gueco et al., 1999). In many other cases, coliform problems were apparently solved by increasing the disinfectant residual without clearly identifying the origin of the coliforms (Kirmeyer et al., 2000; Norton & LeChevallier, 1997; Holt et al., 1995).

FACTORS AFFECTING COLIFORM SURVIVAL AND GROWTH

Once coliform bacteria are introduced into a distribution system, they can be transported with the bulk flow of water, colonize, and grow in biofilms on pipe surfaces or in deposits. The ability of coliform bacteria to survive and even grow in drinking water biofilms has been demonstrated by many authors in pilot distribution system studies (Camper, 1996; Fass et al., 1996; Jones & Bradshaw, 1996; Camper, 1995; Standridge et al., 1995). Regrowth of coliforms was also evidenced in a

Table 2 Factors related to increased coliform occurrence in the Seattle, Wash., distribution system*

Factors	Effect on Coliform Occurrences
Lack of adequate chlorine residual in distribution system pipes	Inadequate disinfection of the coliform bacteria
No filtration of the source water	May increase the passage of coliform from the water source into the system, allowing coliform attached to particulate matter to pass through the disinfection barrier
Presence of uncovered reservoirs	Allows water contamination (particularly from birds and animals); increases the water residence time in the system
Presence of numerous storage tanks	May result in long water residence time, so that particulate matter harboring coliforms tends to settle and accumulate in the tanks
Inadequate flushing program (dead ends only)	Protection of the coliforms in pipe biofilm
Corrosion-control program only initiated in 1982	Protection of coliforms in corrosion by-products, particularly in unlined cast-iron pipes

*From Oliver and Pimentel, 1998

full-scale distribution system (Edberg et al., 1994; LeChevallier et al., 1987). The issue of coliform regrowth in drinking water has previously been reviewed by LeChevallier (1990).

Water Temperature

Water temperature affects all processes involved in microbiological water quality: microbial growth rate, disinfection efficiency, decay of disinfectant residual, corrosion rates, and distribution system hydraulics (increased water velocity from increased consumer demand) (LeChevallier, 1990). In most of the reported cases, more coliform occurrences have been noted in distribution systems during summer months when water temperatures are at their highest (Olstadt et al., 1998; Colbourne et al., 1991; Wierenga, 1985; Hudson et al., 1983). Many authors (Besner et al., 2001; Volk & LeChevallier, 2000; LeChevallier et al., 1996b; Volk & Joret, 1994; LeChevallier et al., 1991) have associated coliform growth with water temperatures higher than 15°C (59°F).

Type and Concentration of Disinfectant

Maintenance of a chlorine residual throughout the distribution system is usually recommended to minimize bacterial growth and coliform occurrence. However, some European utilities located in the Netherlands, Germany, Switzerland, and France are successfully producing and distributing hygienically safe and biologically stable drinking water without a disinfectant residual (Klein & Forster, 1999; Van der Kooij et al., 1999; Fokken et al., 1998; Lévi et al., 1992). The application of multiple treatment barriers, the production of biostable water, and the use of biostable materials for distribution infrastructures—as well as the application of protective measures to prevent recontamination in the distribution system—permit such a practice (Te Welscher et al., 1998; Van der Kooij et al., 1995). In North America, as well as in the United Kingdom, the use of disinfectants remains the favored approach to control microbiological water quality in the distribution system (Trussell, 1999).

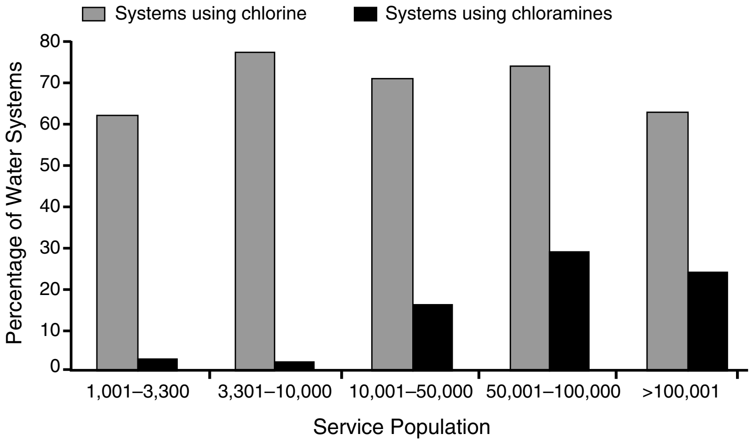
Although chlorine is less efficient for the inactivation of biofilm bacteria than for bacteria present in bulk water (Parent et al., 1996; Servais et al., 1995; Mathieu et al., 1992; Paquin et al., 1992; Van der Wende & Characklis, 1990), relatively low chlorine concentrations have, in some cases, been successful in controlling the presence of coliform bacteria in a distribution system. Free chlorine thresholds varying from 0.05 to 0.5 mg/L in full-scale distribution systems have been quite useful in keeping the rate of occurrence of coliforms low (Besner et al., 2001; Volk & LeChevallier, 2000; Kiéné et al., 1999; LeChevallier et al., 1996b; Volk & Joret, 1994), especially when combined with a good biological stability of the treated water (Gatel et al., 2000). A pilot distribution system study (Parent et al., 1996) showed that as soon as chlorine could be detected in the system under study (0.1 mg/L), it inactivated the coliforms in the water phase. The only question at this time is whether the disinfectant really inactivates the coliforms or if they are still viable but just nondetectable by cultivation on selective media (injured coliforms). This uncertainty is undoubtedly promoting the development of alternative methods for detecting coliform bacteria in water samples. In contrast, many authors have reported that even high chlorine doses are ineffective in controlling coliform occurrence. Coliform-positive samples have been reported to occur in distribution systems with free chlorine residuals ranging from 0.6 to 4.0 mg/L (Schreppel & Geiss, 1996; Norton et al., 1995; LeChevallier et al., 1987; Ludwig et al., 1985; Wierenga, 1985; Martin et al., 1982). Even though increasing chlorine residuals has helped to control coliform occurrence in some cases (LeChevallier, 1990; Hudson et al., 1983), such chlorine increases may not always be an acceptable solution because of the formation of potentially carcinogenic disinfection by-products (DBPs) (Marret & King, 1995).

Because the use of chlorine has several drawbacks (taste, odor, halogenated compound formation) and does not totally guarantee a coliform-free drinking water, some utilities have looked at alternative disinfectants, such as chloramines, to control microbiological quality in the distribution system (Figure 1). Monochloramine is generally considered to be a less effective disinfectant than free chlorine because higher concentrations are needed for a similar level of bacterial inactivation (Mathieu et al., 1992). However, because chloramine is a less reactive disinfectant, it is more stable than free chlorine and persists longer in the distribution system. Its use is therefore beneficial in water networks where it is difficult to maintain a free chlorine residual because of corrosion or high water residence times (Song et al., 1998; Colbourne et al., 1991). However, many utilities first switched to monochloramine as a secondary disinfectant in order to comply with DBP regulations rather than to control total coliforms (Kreft et al., 1985; Mitcham et al., 1983).

The use of monochloramine also proved, in many cases, to be more effective than free chlorine in controlling coliform growth in bench-scale units (Momba et al., 1999; Camper et al., 1997) and in full-scale distribution systems (Norton & LeChevallier, 1997; LeChevallier et al., 1996b). The combination of monochloramine stability and its superior ability to penetrate biofilm has resulted in both greater efficiency with respect to attached organisms and better coliform control (LeChevallier et al., 1990). However, the use of monochloramine does not necessarily represent the perfect solution to coliform occurrence in the distribution system because it is less efficient than free chlorine in controlling a sudden pulse of contamination (Snead et al., 1980) and can lead to nitrification episodes (Wilczak et al., 1996).

Bacterial Nutrients

In drinking water, the limiting nutrient for the growth of heterotrophic bacteria is usually the biodegradable fraction of dissolved organic carbon (DOC), even though the limitation of bacterial growth because of the availability of phosphorus has also been shown in some distribution systems (Miettinen et al., 1997; Sathasivan et al., 1997). This biodegradable fraction is usually expressed in terms of assimilable organic carbon



Source: USEPA, 1997

Figure 1 US community surface water systems using monochloramine as a secondary disinfectant

(AOC) or biodegradable DOC (BDOC) concentrations, reflecting two types of analytical procedures (Huck, 1990). Studies have shown that AOC (LeChevallier et al., 1987) and BDOC (Servais et al., 1995; Servais et al., 1992) generally decrease with increased residence time of the water in the distribution system. It was hypothesized that such a decrease is caused by carbon utilization by bacteria and subsequent growth. Thus, bacterial regrowth may be limited by decreasing the AOC and BDOC content of the water leaving the plant.

Several attempts have been made to define the threshold concentration of biodegradable organic matter below which water can be considered biologically stable. One study (Van der Kooij et al., 1989) proposed an AOC value of 10 µg/L, and another (Servais et al., 1995) suggested a BDOC value of 0.15 mg of carbon (C) per liter. Coliform occurrences in the distribution system have also been associated with higher water biodegradable organic matter content from AOC greater than 50 (LeChevallier et al., 1991) or 100 µg/L (Volk & LeChevallier, 2000; LeChevallier et al., 1996b) and BDOC consumption in the distribution system higher than 0.15 mg C per liter (Volk & Joret, 1994). The benefits generated by reducing the biodegradable content of treated water through biological treatment have been demonstrated for the microbiological quality of water (Prévost et al., 1998) and for biofilm density (Volk & LeChevallier, 1999). Similarly, Laurent et al. (1999) noted a reduction in the number of positive coliform samples in a distribution system following the installation of a nanofiltration treatment. Finally, certain lubricating oils were shown to release substantial levels of AOC into the water, which could in turn lead to increased bacterial growth (Sidorowicz, 1996; White & LeChevallier, 1993). Oil for lubricating well pumps has been associated with the occurrence of coliforms in community wells operated by a US facility (White & LeChevallier, 1993).

Pipe Corrosion and Corrosion Control

The intensity of bacterial colonization of distribution system pipes is influenced by pipe material characteristics (Niquette et al., 2000; Verran & Hissett, 1999). It has also been shown (LeChevallier et al., 1993) that corrosion has a major impact on biofilm persistence in a chlorinated distribution system and that both the type and the rate of corrosion could affect the ability of chlorine to inactivate biofilm bacteria, including coliforms. Coliforms have been detected in corrosion tubercles (Clement et al., 1998; Emde et al., 1992; LeChevallier et al., 1987; Martin et al., 1982). A positive relationship has been found to exist between the number of miles of unlined cast-iron pipe in filtered, free-chlorinated distribution systems and coliform occurrences in the study of LeChevallier et al. (1996b), suggesting that the corrosion of iron pipe surfaces is an important factor affecting coliform occurrences.

Introduction of corrosion-control mechanisms may therefore help to improve the effectiveness of chlorine disinfection, favor the presence of a chlorine residual, and enhance control of bacteria. Common mechanisms of corrosion control include increasing pH, remineralization, and the addition of phosphates or silicates. A higher pH decreases metal solubility and reduces surface corrosion. Such treatment has been successful in controlling the growth and occurrence of *Klebsiella pneumoniae* in a chlorinated distribution system (Martin et al., 1982). Pilot-scale studies (Volk et al., 2000; LeChevallier et al., 1993) showed the beneficial use of phosphate-based corrosion inhibitors to minimize the corrosion of iron pipes and its subsequent increased effectiveness on chlorine disinfection, allowing a better control of distribution system biofilms.

In a study of 31 US water utilities, LeChevallier et al. (1996b) also found that the use of phosphate-based corrosion inhibitors was associated with lower coliform levels. In fact, coliform levels were 36% lower in free-chlorinated distribution systems with phosphate levels greater than 0.1 mg/L than in distribution systems with lower phosphate concentrations. The reduction in coliform occurrences following application of corrosion inhibitors can be explained by the fact that corrosion inhibitors reduce three things: the microbial habitat by reducing the mass of corrosion products on pipe surfaces, the adsorption capacity of iron oxides, and the disinfectant demand of existing corrosion products and the pipe surfaces (Abernathy & Camper, 1997).

Sediment Accumulation in Pipes

Particulate matter in a distribution system may originate from various sources, including incomplete removal of particles from raw water, release of fines from filters, precipitation of metal oxides or calcium carbonate, external contamination in pipes and reservoirs, postflocculation, biological growth, and corrosion. The sedimentation of suspended particles to form loose deposits may take place in areas with low-flow conditions (particularly in dead ends or reservoirs). The presence of coliforms in settled material in distribution systems has been shown in several studies (Lu et al., 1997; Oliver & Harbour, 1995; Schreiber & Schoenen, 1994; DeRosa, 1993). To remove sediments from water pipes, flushing programs may be undertaken by water utilities. Unidirectional flushing of the distribution system (Figure 2), in which specific sections of mains are closed in an organized and sequential manner so that water flows in a single direction from a clean to a dirty area, has been reported to generally improve water quality and diminish coliform occurrences (Antoun et al., 1999; Oliver & Pimentel, 1998).

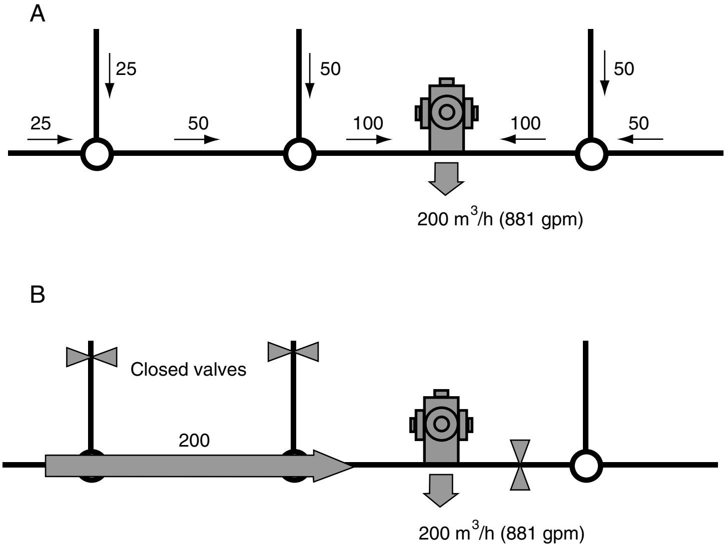


Figure 2 Difference in flow repartition during conventional (A) and unidirectional (B) flushing

MODELING THE OCCURRENCE OF COLIFORMS

The development of coliform predictive models has been attempted to predict, both in time and space, coliform regrowth in the distribution system. Four predictive models (shown in Table 3) have been found in the literature and compared. These models are a combination of differential equations and/or statistical or probabilistic approaches. They usually include the influence of classical water quality parameters known to affect coliform regrowth, such as water temperature, total count of bacteria, residual chlorine, and organic matter. However, because it is known that the occurrence of coliforms in a distribution system does not result from regrowth alone, some authors (Gauthier et al., 1999b; Fang et al., 1997) have proposed the use of an alternative approach using a dynamic visualization/data-based approach that takes into account water quality data as well as many other types of parameters, such as treatment breakthrough and distribution system intrusion, to explain coliform occurrence.

Coliform Predictive Models

Model based on threshold values

The AL.COL (Alerte coliformes) model of Volk and Joret (1994) was developed to predict coliform occurrences in the full-scale distribution systems of the Parisian suburbs, and it is based on weekly water quality monitoring data from five network sampling sites. As shown in Table 3, threshold values associated with an increased frequency of coliform occurrence have been established to define a level of risk at each sampling site resulting from the combination of the threshold values exceeded each week. Application of the model has shown that coliform occurrences were often preceded by an elevated risk (three or four positive criteria) at least one week before the coliform occurrence. During the whole study, the probability of coliform occurrence varied between 26% and 63% when the four criteria were positive, and the probability was less than 1% when none of the thresholds was exceeded.

Log-logistic autoregressive multivariate model

Further studies of the Parisian suburbs' distribution systems led to the development of a log-logistic autoregressive multivariate model for the prediction of coliforms (Gatel et al., 1995). Using the same water quality parameters as the AL.COL model, the authors collected 40 to 60 samples

Table 3 Coliform predictive models

	Volk & Joret, 1994	Gatel et al., 1995	LeChevallier et al., 1996a	Gale et al., 1997
Model parameters	Total coliforms; temperature (T°); total bacteria count estimated by epifluorescence microscopy (TDC*); total chlorine residual; consumption of biodegradable dissolved organic carbon between water treatment plant outlet and sampling point (ΔBDOC)	Total coliforms; temperature (T°); total bacteria count estimated by epifluorescence microscopy (TDC); FRC; consumption of BDOC between water treatment plant outlet and sampling point (ΔBDOC)	Total coliforms; AOC; TOC; turbidity; temperature; ammonia; nitrate; phosphate; alkalinity; hardness; disinfectant type and concentration at plant effluent, midpoints, or dead ends of the system; code for filtered system or not	Total coliforms; THB—detected by pour-plate method with yeast-extract agar incubated for 24 or 48 hr at 37°C or 72 hr at 22°C; temperature; total chlorine residual; color, turbidity, TOC, and metallic ions (infrequent records)
Prediction type	Level of risk—no risk: 0 or 1 threshold value exceeded; slightly exposed: 2 or 3 threshold values exceeded; exposed: 4 threshold values exceeded	Expected percentage of positive coliform samples	Definition of predictor variables for coliform occurrences	Overall number of coliform-positive samples

Table continued next page.

Table 3 Coliform predictive models *(continued)*

	Volk & Joret, 1994	Gatel et al., 1995	LeChevallier et al., 1996a	Gale et al., 1997
Model type	Threshold values: T° > 15°C; ΔBDOC > 0.15 mg/L; TDC > 5.2 log; total $Cl_2 < 0.10$ mg/L	Log-logistic autoregressive multivariate model: $\pi_{t+1} = f(\alpha\pi_t + \beta X_t, \lambda)$ in which π_{t+1} is the proportion of coliform-positive samples for week t+1, α is the autoregressive coefficient, X_t is the variable values at time t (FRC, T°, TDC, ΔBDOC), β is the 5 coordinates vector (β_{FRC}, $\beta_{T°}$, β_{TDC}, $\beta_{\Delta BDOC}$, β_m) with β_m corresponding to a constant for the average effect, λ is the structural parameter allowing variations between log and log-logistic functions	Tree-based statistical x models: classification trees for unfiltered and filtered systems; regression tree for coliform occurrences	Generalized linear modeling method: $\text{Ln}\,(p/[1-p]) = \alpha_0 + \alpha_1 \times$ Temperature $+ \alpha_2 \times$ [Total chlorine] $+ \alpha_3 \times$ [THB] in which p is the probability of coliform detection in a 100-mL volume and α_0, α_1, α_2, and α_3 are the constants whose values are estimated from the data

Table continued next page.

Table 3 Coliform predictive models *(continued)*

	Volk & Joret, 1994	Gatel et al., 1995	LeChevallier et al., 1996a	Gale et al., 1997
Time of prediction	Following week	Following week	NA[†]	Per year
Area of prediction	At specific sampling sites	Over the entire distribution system	For the overall group of 31 distribution systems	Distribution system zones

[*] TDC—total direct count of bacteria, BDOC—biodegradable dissolved organic carbon, FRC—free chlorine residual, AOC—assimilable organic carbon, TOC—total organic carbon, THB—total heterotrophic bacteria

[†] NA—not applicable

per week from March to September for a four-year period (1992–1995) to develop the model. To predict the weekly percentage of coliform-positive samples on the network under study, the authors used a log-logistic autoregressive multivariate model (Table 3). The α, β, and γ parameters were estimated from the weekly values and determined using a Metropolis-Hastings–type algorithm. Results showed that the model was strongly influenced by the total direct count of bacteria, whereas temperature and chlorine appeared to be less important factors and the consumption of organic matter could be eliminated. Model predictions (expected percentage of coliform-positive samples) for the weekly data of 1992 and 1994 were shown to correlate well with the observed values, with an R^2 coefficient of 0.75.

Tree-based statistical models

One study (LeChevallier et al., 1996a) proposed the use of a tree-based statistical model to describe how a set of predictor variables could relate to coliform occurrences in the distribution system. Classification tree models were used for unfiltered and filtered water systems, and a regression tree model was used for the general occurrence of coliforms. The tree models were developed from sampling data from 31 US water systems collected every two weeks for an 18-month period. As shown in Table 3, a larger number of variables was used to model the presence or absence of coliforms in comparison with the other existing models. The model resulted in the inclusion of a filtration step, dead-end disinfectant residual, and temperature variables. The classification rule resulted in few false positives (i.e., samples predicted to be coliform-positive but that were in reality coliform-negative), but many false negatives (i.e., samples predicted to be coliform-negative but that were in reality coliform-positive). A separate classification tree model for filtered systems was attempted, but the occurrence of coliforms in filtered systems was not frequent enough to apply an accurate statistical model using the current methodology. A regression tree model for coliform occurrence was also developed. This model showed that in general the highest coliform occurrence rate (13.3% of positive samples) was obtained when phosphate levels (related to corrosion control) were low, the level of alkalinity was low, and the temperature was high.

Generalized linear modeling method

A regression model has been proposed by Gale et al. (1997) with data collected from 13 UK water companies. The companies (divided into distribution zones) provided statutory water quality monitoring data for one or more years between 1990 and 1994 (Table 3). The authors of the study used a generalized linear modeling (GLM) method to determine the probability of coliform detection in a 100-mL water volume. However, following the validation process, because the predicted probabilities of coliform occurrence in 100-mL water samples were very low, the overall number of coliform-positive samples for groups of related zones (according to type of disinfectant used, water source, and treatment train) per year was rather established as the predicted parameter. As observed from the validation of some companies' models (between years and between similar zones in a company's distribution network), the number of positive coliform samples predicted over distribution zones was generally close to the number observed.

Advantages and limitations of the coliform predictive models

Each modeling technique used to predict coliform occurrences has its advantages and limitations. The determination of threshold values (Volk & Joret, 1994) is basic and may be considered quite simple and easy for water utilities to apply. However, the use of statistical methods for coliform prediction probably provides models that are more soundly based. The GLM method used by Gale et al. (1997) is the only methodology able to identify when a model is incomplete, i.e., when modeled parameters (chlorine and total heterotrophic bacteria) are not able to fully explain differences in coliform occurrences between zones of distribution, and that further zonal information is required. Because the GLM method showed that many water company models were incomplete, it emphasized the need to consider factors other than water quality parameters to explain coliform-positive samples. The major strength of the model by Gatel et al. (1995) is the inclusion of an autoregressive component that allows the prediction of the coliform-positive samples for the week to come by using the information available for the prior week and the prediction that was made for that time period. This model is the only one to take previous predictions into account.

Many limitations are associated with the use of these models. Predictions in time may vary from weekly predictions (AL.COL and log-logistic autoregressive multivariate models) to yearly predictions (Gale et al., 1997). As for the tree-based model, the authors themselves found that it clearly lacked the statistical power needed for prediction (LeChevallier et al., 1996a). Specific distribution system locations (sampling points) likely to experience coliform-positive samples are often missing. Apart from the AL.COL model, spatial prediction concerns distribution zones within a water company (Gale et al., 1997) or a whole distribution system (Gatel et al., 1995) assuming homogeneity of the variables throughout the distribution system, which is surely too simplistic. As noted by LeChevallier et al. (1996a) following their examination of commonalities among 31 water systems, better predictive models could probably be obtained by examining data for individual systems. All those models predict only the presence/absence of coliforms in samples; no information is given with respect to concentration values.

Limitations associated with these models may be related to the poor content of coliform databases, because coliform occurrence rates in distribution systems are low and traditional coliform detection methods provide zero or near-zero values most of the time (Gale, 1996). Furthermore, water quality data used for model construction are usually not collected for this specific purpose, which may result in infrequent records for some parameters. Consequently, modeling coliform occurrence is quite a difficult task. The application of those models is also specific to the zones or to the distribution system where they have been calibrated, meaning that parameter optimization is necessary for each zone/distribution system with respect to local operating conditions. Few data are available on the subsequent application of those models by water utilities. In a recent study, Volk and LeChevallier (2000) proposed a variation of the AL.COL model based on three parameters (temperature, AOC, and dead-end disinfectant residual) collected for a group of distribution systems. This led to a probability of coliform occurrence of 16.1% when the three criteria were exceeded. However, it was found that the model had a low level of prediction accuracy and that further refinements were necessary to obtain more reliable results.

Dynamic visualization/data-based approach

Because water quality parameters are clearly insufficient to explain coliform occurrence in a distribution system and the parameters affecting regrowth are not completely known, identified, and measured, modeling approaches seem to have hit a wall. This has led to the use of a dynamic visualization/data-based approach to better understand water quality variations. Such an approach combines hydraulic/water quality simulation models and geographic information systems (GIS-type software), allowing the user to account for numerous parameters such as distribution system structure, operation, and maintenance as well as water quality information. The only limitation of this approach is that its effectiveness is dependent on the extent of the available databases (Barcellos, 2000; Gauthier et al., 1999b; Boulos et al., 1997; Fang et al., 1997; Pena, 1995; Deininger et al., 1992). Fang et al. (1997) reported the development of a program put in place by the City of New York that integrates routine monitoring activities, hydraulic modeling, GIS software, data analysis, and special local investigations to identify and facilitate coliform problem remediation. This program has resulted in a significant decrease in coliform incidence since 1994.

Besner et al. (2001) and Gauthier et al. (1999b) have used an approach that combines structural, operational, and water quality parameters (Figure 3), and they were able to associate coliform recurrence in a section of the Montréal, Que., distribution system with the contamination of a large-diameter pipe with low water velocity, where breaks were reported in 1995. Improvements consisting of the addition of a visualization/exploration tool for database queries have been made to this approach by Besner et al. (2000). This was done to facilitate the study of the impact of distribution system interventions on water quality.

Because coliform occurrence may be triggered by various factors and these factors are not easily quantifiable, it seems unrealistic to try to predict short-term, localized contamination. It is thus more important to improve our understanding of coliform occurrence so that we will eventually be able to quantify the parameters involved. The consideration of multiple databases—combined with their visualization—is a key element in this process.

Modeling Perspectives

The use of artificial neural network (ANN) models could constitute a new research avenue for coliform prediction. ANNs have the capacity to learn from examples and to generalize to previously unseen data, as well as the ability to handle nonlinear relationships. At this time, this type of model has not been used extensively in the field of drinking water treatment and distribution. Rodriguez and Sérodes (1996) used an ANN modeling approach to estimate the disinfectant dose adjustments required during water rechlorination in storage tanks, and Skipworth et al. (1999) predicted the oxidation reduction potential at single and multiple points within a distribution system. Moreover, using ANNs for coliform prediction would require large databases for building strong models, and, as long as coliform data show values below detection levels or zero values, ANN model development will be difficult. In fact, this is probably true for any type of model. As an alternative, the use of atypical bacteria (i.e., noncoliform bacteria on m-Endo agar) as a supplemental indicator group could be considered, as suggested by Brion et al. (2000). That study successfully used atypical bacteria and ANNs for pollution ranking and

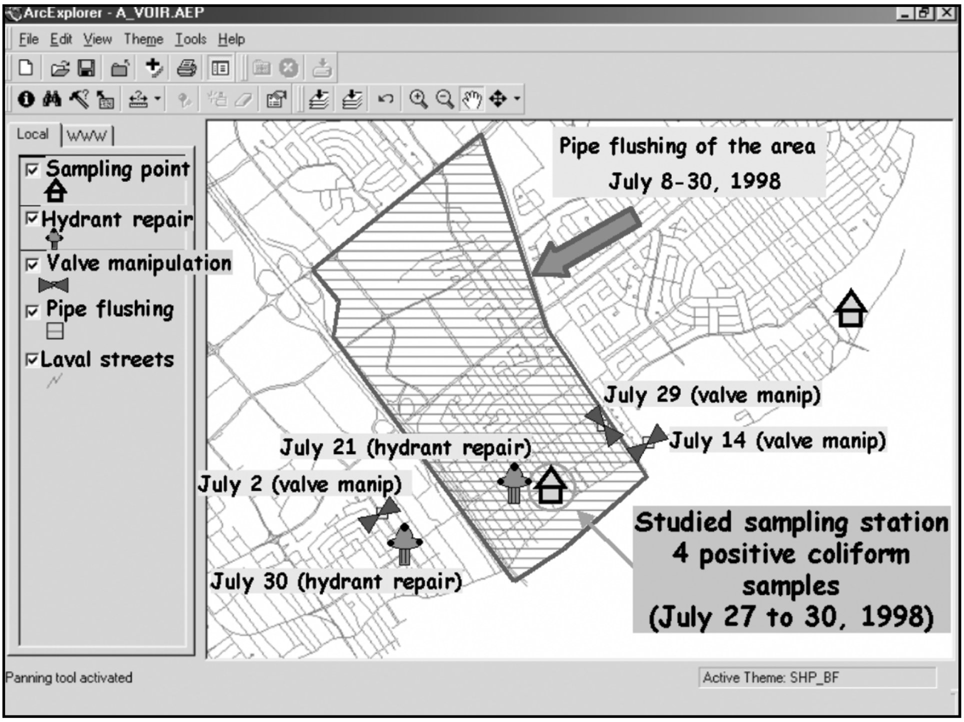


Figure 3 Use of geographic information system–type visualization tools to help explain water quality variations

source identification in a surface water reservoir. The development of more "sensitive" coliform detection methods that would yield other than zero values could also help support the building of such models.

SUMMARY AND CONCLUSIONS

Obviously, distribution systems play a major role in maintaining the quality of treated water. Detection of coliform organisms in a distribution system may result from multiple factors and not only from the type of contamination (treatment breakthrough/intrusion). Coliform occurrence may be greatly affected by the following distribution system features:

- The hydraulics influence the transport of contaminants, the deposition/resuspension of particulate material, the residence time of the water in the system, and the occurrence of low pressure.

- Physical characteristics, such as elevated water temperatures, were shown to often be related to coliform detection.
- Chemical characteristics, such as the type and concentration of disinfectant used in the system and the level of biodegradable organic matter, have also been related by many authors to coliform occurrence.
- Structural properties, such as pipe material, may constitute the principal influence on the degree of corrosion and the rates of pipe breakage and leakage that will be found in the system.

The effects of some of these factors on coliform occurrence are common to all distribution systems, whereas others are more site-specific. However, taken as a whole, all these factors will affect the capability of coliforms to survive in a distribution system. Because of the transport/deposition/survival mechanisms occurring in the distribution system, the signal of bacterial contamination entering the system may consequently be biased, so that it becomes very difficult in some cases to detect contamination directly at its point of entrance or at the time it penetrated the system. Such situations imply that water utilities and authorities are probably failing to detect numerous contamination events in their systems and that those that are detected are often difficult to explain.

Understanding the origins of coliform occurrence in a distribution system, which are usually not very obvious, is the first step toward a permanent solution to the problem. In some cases, improvements in water treatment and network management have allowed water utilities to solve their problems without determining the source of coliforms in their systems (Clark et al., 1999). The application of measures such as reducing the nutrient level entering the distribution system, reducing water residence time in the network, applying a chloramine residual, as well as methods to control pipe corrosion and unidirectional flushing, have shown in many cases to be quite effective in reducing coliform occurrence (Kirmeyer et al., 2000; USEPA, 1999).

Replacing total coliforms by *E. coli* as the indicator, as proposed by Edberg et al. (2000), would perhaps lead to a stronger link between health risk and regulatory failure. Yet it also would result in a major reduction in coliform-positive events, further reducing the potential for determining the source of the fecal contamination. Moreover, predictive models for failures would be even more difficult to build because of the small number of positive samples. In such a case, total coliforms could be kept as the operational parameter used by utilities to evaluate regrowth phenomena or nonfecal contamination while being removed from the assessment of compliance issues (Stevens et al., 2001).

The prediction of noncompliant samples seems difficult, if not impossible, because the number of possible causes for coliform occurrence is high and coliform detection is biased by transport and survival mechanisms in the distribution system, which are difficult to quantify. Therefore, greater understanding of coliform occurrence is needed to eventually enable quantification of the parameters involved. Because the tools currently available are not adequate, the development of new approaches, which should involve the consideration of multiple databases, is essential. This is critical because water quality information is necessary, but often not wholly sufficient, to explain coliform occurrence in a distribution system. All types of databases (operation and maintenance, hydraulic, and so forth) should be made available to increase the chances of identifying coliform problems. Because our capacity to explain and predict is also strongly influenced by the results obtained by the coliform detection methods that "unfortunately" give zero values in 90% to greater than 99% of the cases, it is also suggested that new methods allowing for higher values be accepted with a view to understanding, but not regulating, coliform presence. The combination of a more effective detection method and an analytical tool integrating a variety of parameters should greatly help water utilities in identifying the causes of water quality problems and, consequently, allow them to apply corrective measures to eliminate coliform occurrence from their distribution systems.

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CHAPTER · 3

Effect of Distribution System Materials and Water Quality on Heterotrophic Plate Counts and Biofilm Proliferation

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BACKGROUND

Biofilms on pipe walls in water distribution systems are of interest since they can lead to chlorine demand, coliform growth, pipe corrosion, and water taste and odor problems. The study described in this chapter is part of an AwwaRF and Tampa Bay Water tailored collaboration project to determine the effect of blending different source waters on distribution system water quality. This project is based on 18 independent pilot distribution systems (PDS), each being fed by a different water blend (seven finished waters blended in different proportions). The source waters being compared include groundwater, surface water, and brackish water. These are treated in a variety of pilot distribution systems, including reverse osmosis (RO) (desalination), both membrane and chemical softening, and ozonation with biological activated carbon (BAC) for a total of seven different finished waters. The observations from this study have consistently demonstrated that unlined ductile iron was more heavily colonized by biomass than galvanized steel, lined ductile iron, and PVC (in that order) and that fixed biomass accumulation was more influenced by the nature of the supporting material than by the water quality (including secondary residual levels). However, bulk liquid water cultivable bacterial counts (i.e., hetetrotrophic plate counts, or HPCs) did not increase with greater biofilm accumulation, but results to date suggest high HPCs correspond with low disinfectant residual more than high biofilm inventory. Temperature affected biofilms also, and AOC was important when residual was between 0.6 and 2.0 mg Cl_2/L. An additional aspect of the work is that the potential of exoproteolytic activity (PEPA) technique was used along with a traditional so-called destructive technique in which the biofilm was scraped off the coupon's surface, resuspended, and cultivated on R2A agar. Both techniques gave similar trends and relative comparisons among PDSs but culturable biofilm values were several orders of magnitude lower than PEPA values.

INTRODUCTION

One of the major concerns of drinking water producers is to provide bacteriologically safe water to the public. Controlling bacterial growth in North America largely depends upon maintaining disinfectant residual throughout the distribution systems. The disinfectant role is to provide residual in distribution systems to control microbiological degradation of water quality and protect against possible contamination. The biofilm can harbor coliform organisms, as well as viruses and opportunistic pathogens like *Cryptosporidium* oocysts [1]. A high bacterial population in potable water can be associated with an increased possibility of waterborne disease, taste and odor problems, corrosion, and the need to maintain a higher disinfectant residual [2]. Moreover, coliform bacteria have been associated with a high abundance of heterotrophic bacteria and biofilm, increasing the potential for health risk [3]. It is also thought that suspended cells in the liquid phase originate from the biofilm through detachment processes during the distribution of drinking water and that control of suspended cell concentrations is assisted by minimizing biofilm cells. However, biofilm-associated microorganisms have been shown to be less susceptible to disinfectants than their planktonic counterparts [4,5]. The increasing use of monochloramines in North America in order to prevent production of harmful disinfection by-products is an interesting alternative to free chlorine to control bacterial growth since monochloramines are thought to penetrate deeper into a biofilm matrix [6].

In the past, controlling bacterial regrowth in distribution systems has focused on limiting nutrient levels (AOC and BDOC) and use of secondary residual (free and combined chlorine). The effect of distribution system materials on heterotrophic plate counts and biofilm proliferation has been more extensively studied during the past decade. Recent evidence suggests that biofilm growth is closely associated with corrosion of pipe materials, through increase of disinfectant demand by corrosion products. Consequently, the corrosion potential of pipe materials influences bacterial regrowth in distribution systems and ultimately downstream water quality [7]. Recent study also shows that densities of fixed bacterial biomass were dependent upon pipe material and that gray iron supported more biofilm cells than plastic-based materials in different oligotrophic drinking water environments [8]. It was concluded that bacteria fixed on gray iron appeared to be more protected from the chlorine residual in the water than those fixed on noncorroding materials, implying that pipe reactivity is an important promoting factor of bacterial regrowth in the distribution system. Therefore, pipe material seems to have a strong influence on bacterial regrowth. This chapter investigates the interrelationships of HPC and biofilm proliferation.

MATERIALS AND METHODS

Pilot Distribution Systems

This study was part of an AwwaRF and Tampa Bay Water tailored collaboration project to determine the effect of blending different treated source waters on the water quality of distribution systems. The project analysis was carried out at the University of Central Florida, Civil and Environmental Engineering Department. There were 18 independent pilot distribution systems (PDSs), each being fed by a different water blend (seven finished waters blended in different proportions). The source waters being compared include groundwater, surface water, and brackish water. These were treated in a variety of pilot distribution systems, including RO (desalination), both membrane and chemical softening, and ozonation with BAC for a total of seven different finished waters. The pilot distribution system (PDS) lines were constructed from actual member governments' distribution systems. The first 14 lines (PDS 1–14) were made of a combination of PVC, lined ductile iron, unlined cast iron, and galvanized steel pipe sections attached in series in that order (upstream to downstream) and are referred to as "hybrid" lines. The four remaining lines (PDS 15–18) were made of a single material (unlined ductile iron for PDS 15, lined ductile iron for PDS 16, PVC for PDS 17, and galvanized steel for PDS 18) and are referred to as "single-material" lines. Each PDS had a hydraulic retention time of about five days. Chloramines were used for primary disinfection and as secondary residual in the PDSs.

Incubation Conditions

Coupons cut from existing pipes excavated from the ground (PVC, lined and unlined ductile iron, and galvanized steel, all from utilities in the

Tampa Bay area) were incubated in these lines for a three-month period (corresponding to a single operational phase of the project). Coupons from each material under study were incubated in the "hybrid" lines, while in the "single-material" lines coupons from the same material as the line were incubated. Coupons, which were taken from existing pipes, were sometimes extensively corroded and were referred to as "aged" coupons. Coupons were always in duplicate. Duplicate coupons for both assays (HPC and PEPA) were provided. Each was 3 cm in diameter and affixed to a peg made from a PVC welding rod. The coupons were then placed inside of a cradle consisting of a 3-in.-diameter PVC pipe cut lengthwise. The cradles had holes for the coupon pegs and were placed inside of a 4-in.-diameter PVC pipe connected to the end of each PDS for incubation of the coupons. Before being placed inside of the cradles, the outer side of the coupons was taped with 1-in.-wide Teflon tape in order to expose only the inner surface of the coupons to the water. The operation of the pipe cradle was similar to that of the PDSs. Specifically, it normally operated at a flow of 4.7 gpd and was flushed at 1 fps once a week.

Sampling Conditions

The coupon holder was carefully removed from the cradles, with the coupons still affixed, and then transported inside of a closed PVC container. The humidity in the container was elevated by placing a wet sponge inside in order to prevent desiccation of the biofilms. Upon arrival in the laboratory the coupons were analyzed during the next 24 hr.

HPC Enumeration of the Biofilm

The coupons were rinsed very carefully with phosphate buffer solution (PBS) twice. The biofilms were detached from coupons manually using a sterile weighing spatula (sterilized by 70% Ethanol) into 4 mL of sterile PBS, and then homogenized using a tissue blender (Tissue Tearor™, Biospec Products, Inc., Racine, Wis., USA) at 3,000 rpm for 2 min. The sample was then serially diluted and two plates per dilution were spread on R2A agar plate (ref. 1826-17-1, Difco Laboratories, Detroit, Mich., USA) and incubated for seven days at 25°C. Typical dilutions used were 10^{-3} and 10^{-4}. Finally, after incubation and enumeration, the results were expressed as cfu/cm^2 by taking into account the dilutions used and the surface area of the scraped coupons. Results obtained with this technique are further referred to as BFHPCs (Biofilm HPCs). This spread plate technique on R2A agar is outlined in Standard Method 9215C [9].

Potential of Exoproteolytic Activity Assay

The protocol used was similar to the one described by Laurent and Servais (1995), except that it was slightly modified to use much less expensive batch reactors for the biofilm coupons [10]. This facilitated high sample volumes at a reasonable cost. The batch reactors were cylindrical PVC cups made from 1¼-in. schedule 40 PVC pipe and a flat 1¼-in. PVC cap for the bottom. The height of the cup was about 3 to 4 in. Upon arrival in the laboratory, the coupons were detached from the coupon holder with caution. New Teflon tape was affixed on the used Teflon tape to prevent any interactions between the bacterial biomass deposited on the outer part of the coupon and the reagent. To avoid desiccation, coupons were analyzed as soon as possible. Both duplicate coupons were then placed in autoclaved PVC containers covered by aluminum foil. When the experiment was ready to be carried out, 8 mL of solution of nonfluorescent L-Leucyl-β-Naphtylamide (LL-βN, Sigma, St. Louis, Mo., USA) at saturating concentration of 1 m*M* was poured into the container to submerge the coupon. The concentration was that used by Somville and Billén (1983) [11]. At this point the enzymatic hydrolysis reaction that releases the fluorochrome β-Naphtylamine from the LL-βN molecule occurred due to the action of bacterial exoenzymes of the coupon biofilm. From this point and at 10-min intervals, 2 mL of subsample was removed and the associated fluorescence was measured at 410 nm under 340 nm excitation. Then the 2 mL was returned to the PVC container. A standard curve correlating fluorescence intensity to βN concentration was required for each set of samples. After conversion using the standard curve, βN concentration was plotted against time and the slope, i.e., the βN concentration increase (nmol of βN produced per min and per cm^2) was obtained by simple linear regression on the linear portion of the response curve. Then it was converted into biomass expressed in μg C/cm^2 by multiplying it by 6.57 (reciprocal of the slope of the correlation straight line established by Laurent and Servais [1995]) [10]. Finally this was converted into cells/cm^2 by considering an average carbon content of 20×10^{-15} g C/cell [10].

Water Quality Analyses

Most water quality analyses for the bulk liquid were carried out with respect to Standard Methods [9]. Residual concentration of free and total chlorine was measured on-site with a portable spectrophotometer (Hach 46700; precision ±0.03 mg Cl_2/L) and recommended reagents (Hach No. 21055-69, N,N-diethyl-*p*–phenylenediamine (DPD); Hach No. 21056-69, DPD and potassium iodide). The sampling bottles and 40-mL vials for AOC analysis were muffled at 525°C for 5 hr after cleaning. AOC was measured using the rapid method of LeChevallier et al. (1993), except that plate counts were used to enumerate bacteria rather than ATP fluorescence, in conjunction with *Standard Methods* 9217 (1995) and the method of van der Kooij (1992) [7,9,12].

RESULTS AND DISCUSSION

The results summarize four operational phases of the TBW-AwwaRF project, i.e., 12 months of study. The observations from this study have consistently demonstrated that unlined ductile iron was more heavily colonized by biomass than galvanized steel, lined ductile iron, and PVC (in that order). Figure 1 presents a representative example of this observation, for the second phase of the study, for the hybrid lines. In the hybrid data the secondary residual was equal for all four coupons in a single hybrid PDS. Thus, in this case it can be seen that cast iron had a

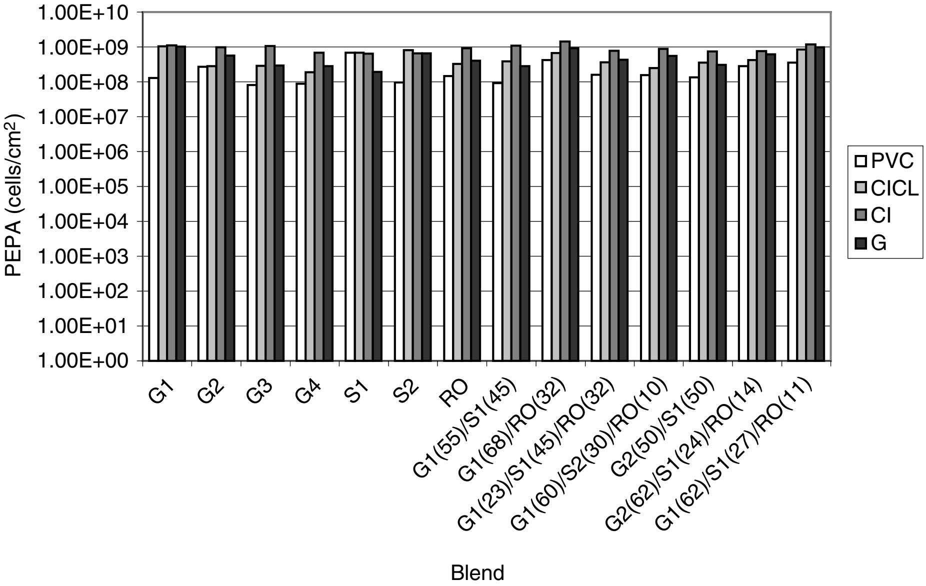

Figure 1 PEPA end of phase 2 (sort by blends)

higher biomass inventory (as much as one order of magnitude) than PVC, even when residual was equal. Holden et al. (1995) documented similar results, comparing biofilm inventories on cast iron and medium-density polyethylene (MDPE) [13]. It can be noted that the hybrid lines were fed with different blends. This did not seem to have an effect on the observed trend. Therefore, fixed biomass accumulation was influenced more by the nature of the supporting material than by the water quality.

Figure 2 presents the combined results of the four phases of the study for the pure lines (fed by same blend but consisting of different single materials). The trend in Figure 2 is the same (e.g., PVC lowest, CI greatest) if average values are compared, even though combined residual levels were different from one line to the next. The fixed biomass inventory was fairly insensitive to residual levels, which were much lower for unlined metals.

In contrast, bulk liquid bacterial counts (i.e., hetetrotrophic plate counts or HPCs) varied inversely with disinfectant residual (Figure 3). The highest HPCs were observed in the unlined cast iron and galvanized steel lines, which had very low residuals.

Analysis of both hybrid and pure PDS data throughout the length of the study showed that combined residual had a significant effect on bulk

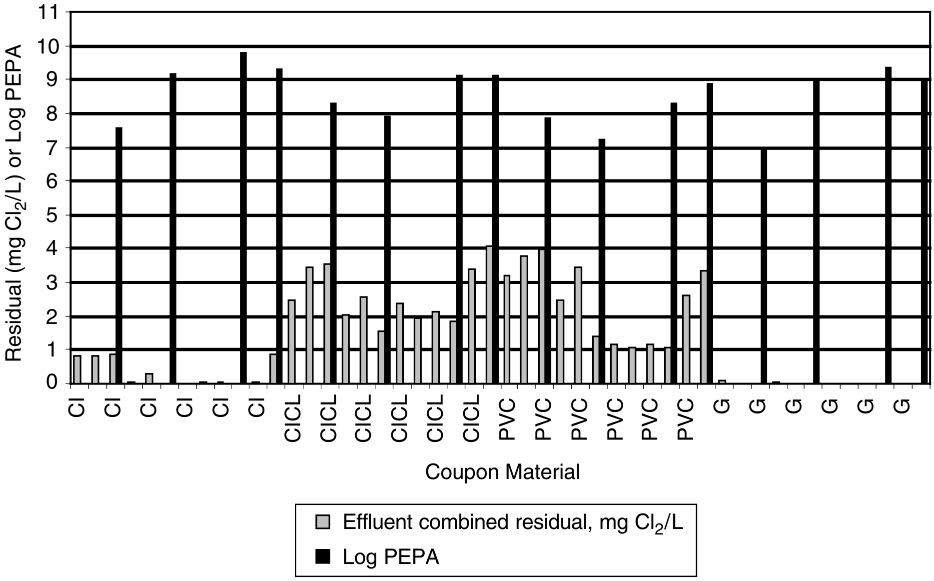

Figure 2 Log PEPA and effluent combined residual vs. material—pure lines only

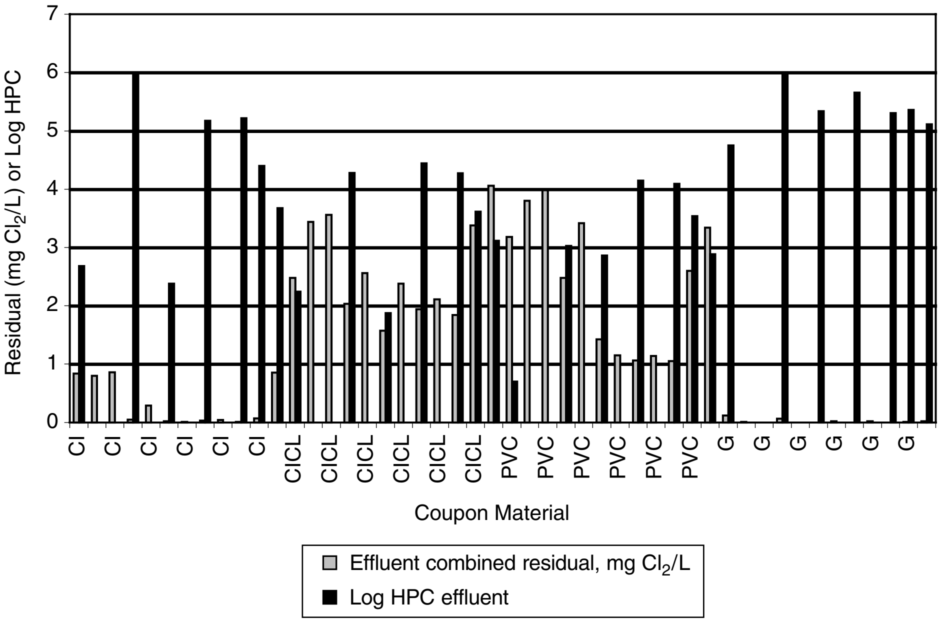

Figure 3 Log HPC and effluent combined residual vs. material—pure lines only

liquid HPCs, particularly when levels were below 0.6 mg Cl_2/L (Figure 4). HPCs above 100,000 cfu/mL were not seen if residuals were above 0.6 mg/L (chloramines were being used), but very low counts (<100 cfu/mL) were sometimes seen in systems with negligible residual. Lower counts were consistently seen with residuals in excess of 0.6 mg/L, however. Below residual levels of 0.6 mg Cl_2/L, HPCs were high (10^5 or greater being common), and this seemed to be true regardless of AOC concentrations or stability.

However, when residuals were between 0.6 and 2.0 mg Cl_2/L, HPC counts correlated directly with AOC stability (and more loosely with AOC concentration). Waters showing changes in AOC levels (between influent and effluent) greater than 37 µg C/L tended to have elevated HPC counts (>10^4 cfu/mL; Figure 5). The data were ambiguous above 2.0 mg Cl_2/L of residual with respect to AOC since there was not a broad range of influent AOC values in that data set, and none above 109 µg C/L. AOC and HPCs seemed stable with residual above 2.0 mg Cl_2/L, but whether that would have been true for a higher AOC finished water could not be evaluated. With biofilm inventory there was no significant difference in

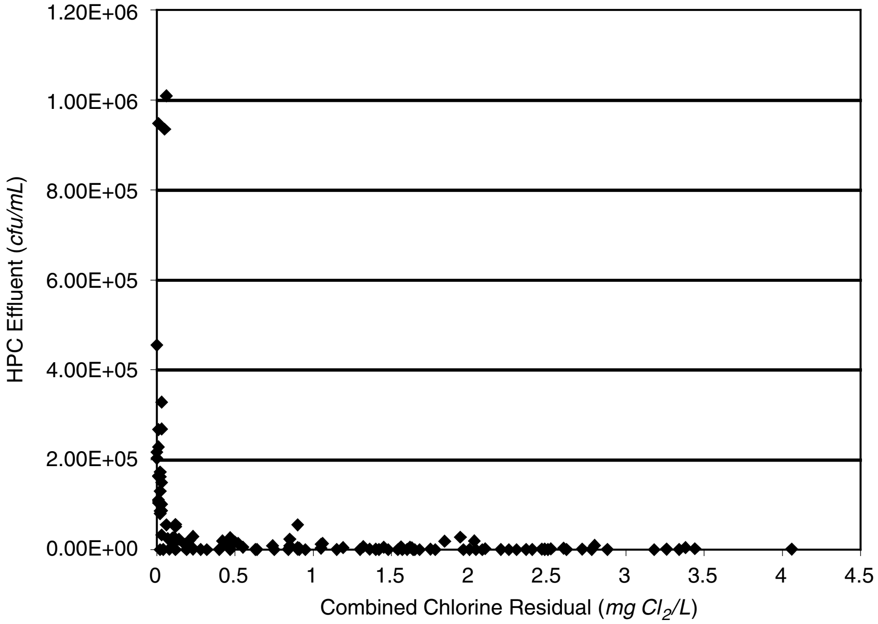

Figure 4 HPC effluent vs. effluent combined residual

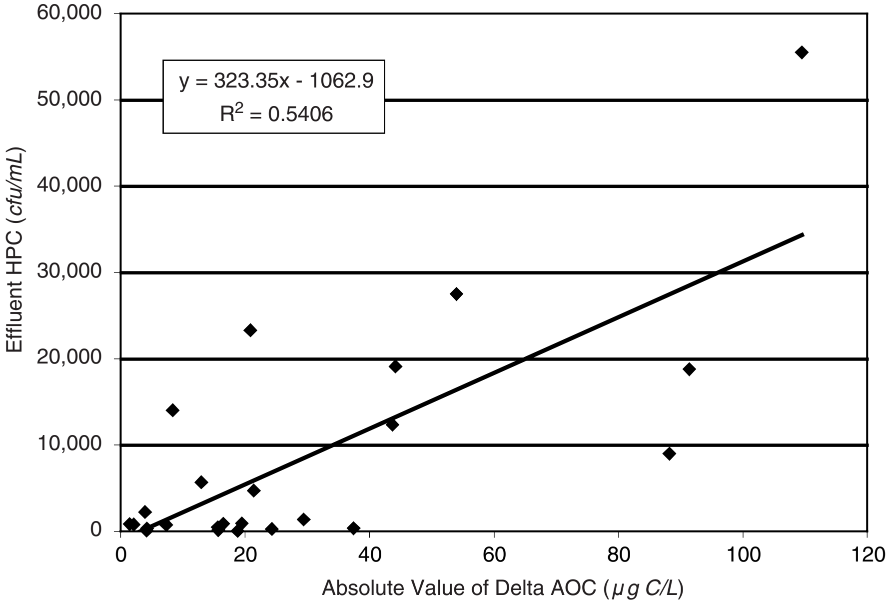

Figure 5 HPC effluent vs. absolute value of delta AOC

the hybrid lines even though HPCs varied from 100 to 100,000 cfu/mL. There did appear to be a correlation in the pure lines, but because of the confounding effect of material and residual concentration this correlation could not be exclusively attributed to either residual or material. Low residuals increased the probability of high HPC counts significantly, while the effect of material on HPC counts was inconclusive, and probably more a result of residual consumption associated with the material.

The effect of temperature was also investigated. It was observed that fixed biomass was sensitive to increase of temperature (Figure 6). An increase of 5°C (from 18° to 23°C) led to increases of about one order of magnitude in biofilm inventory, while increasing from 23° to 26°C multiplied the fixed cell densities by a factor of 5 to 10. The effect of temperature on HPCs was inconclusive, even though depletion of residual was observed at higher temperatures. No straightforward correlation was found connecting temperature and HPCs. However, the impact of higher-temperature driving increased consumption of combined residual levels and affected HPCs in a very significant way.

An additional aspect of the work to date is that in addition to the PEPA technique, the biofilm was scraped off an identical coupon surface for the pure material lines, resuspended, and cultivated on R2A agar (BFHPC). Figure 7 (second phase, representative of the other phases) shows that the BFHPC technique yielded much lower estimates of the biofilm inventory on the coupons (two to four orders of magnitude lower than the PEPA results). However, the trend observed for the effect of material on biofilm accumulation, e.g., that cast iron was more heavily colonized than PVC, was the same using both techniques. This strengthens the conclusion that material was a major factor in determining the biofilm inventory.

Biofilm bacteria are thought to be more resistant to disinfectant due to a physiological state associated with low growth rates or due to increased diffusional resistance created by the extracellular polymers [4,5]. Studies by de Beer et al. (1994) using chlorine-sensitive microelectrodes demonstrated that chlorine did not fully penetrate into the biofilm matrix because chlorine reacted with the biofilm constituents faster than it could diffuse into the biofilm [14]. This phenomenon has also been documented by more recent studies [15,16]. This resistance to disinfectant levels can explain the insensivity of biofilm inventory to combined residual observed in this study.

In terms of bulk liquid HPCs, the effect of materials was most significant in terms of material impact on residual levels. Unlined metals

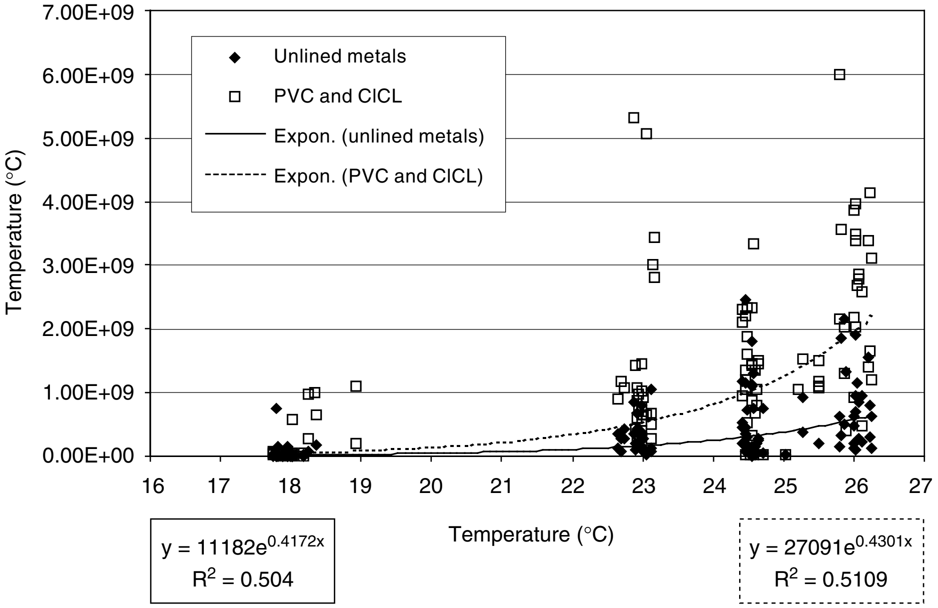

Figure 6 PEPA vs. average temperature (influent/effluent)

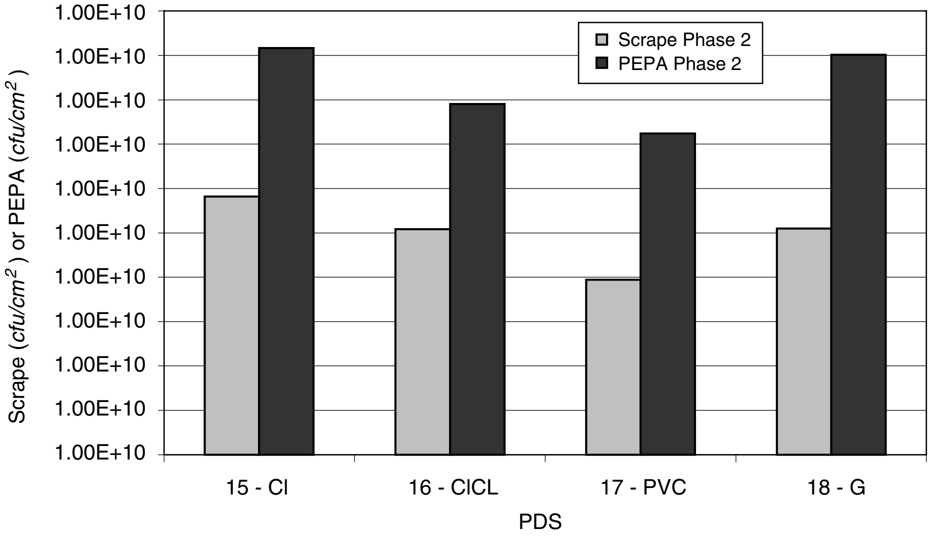

Figure 7 PEPA and BFHPC vs. material—pure lines only

depleted residual levels, and this resulted in elevated HPCs. Any intrinsic effect of material was of a much smaller magnitude than residual effect and was not observable if present.

CONCLUSIONS

The major findings of the experiments discussed herein are summarized below:

- Biofilm inventory was a function of the material and was relatively insensitive to secondary residual levels or to variations in water quality.
- HPC counts could not be directly correlated to biofilm inventory. HPCs were most significantly impacted by residual. As secondary residual decreased from 0.6 to 0 mg Cl_2/L, the probability of high HPC counts (>100,000 cfu/mL) increased very significantly.
- AOC stability affected HPC proliferation when residuals were between 0.6 and 2.0 mg Cl_2/L.
- Unlined metals and higher temperature (summertime) resulted in significantly greater residual consumption, and this increased the probability of high HPC counts if residual consumption was nearly complete.

ACKNOWLEDGMENTS

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CHAPTER · 4

Biostability and Microbiological Quality in a Chloraminated Distribution System

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BACKGROUND

In this study, two components were evaluated: biostability, measured by assimilable organic carbon (AOC) concentration; and microbiological quality, determined by heterotrophic plate counts and coliform counts. The study centered on a full-scale chloraminated distribution system. Results from full-scale monitoring showed that the water in the system was biologically stable and there was no significant microbial activity in the system in spite of a moderate to high AOC concentration of 162 ± 24 μg/L as acetate-carbon. Batch experiment results indicated that the treated water containing monochloramine was biologically stable, and removal of the residual chlorine resulted in rapid consumption of AOC and bacterial regrowth. This study suggested that the treated water's AOC concentration may not be the dominant factor for controlling bacterial regrowth in all distribution systems. Biostability can be achieved in a water supply containing a moderate to high AOC concentration by maintaining a high disinfectant residual throughout the system.

It is well documented in the scientific literature that drinking water quality can deteriorate during passage through distribution systems (O'Connor et al., 1975; Larson, 1966). These changes typically include loss of chlorine residual (Skadsen, 1993) and formation of disinfection by-products (Chen & Weisel, 1998). In addition, potential exists for a variety of microbially mediated processes, including oxidation of organic matter (LeChevallier et al., 1991), nitrification (Skadsen, 1993), sulfate reduction (in reducing microenvironments) (O'Connor et al., 1975), and microbially induced corrosion (Lee et al., 1980). Bacteria are able to grow in distribution systems even in the presence of a disinfectant residual (LeChevallier et al., 1987). This process is termed "bacterial regrowth."

Potential problems resulting from bacterial regrowth include growth of both undesirable organisms (Levy et al., 1986) and opportunistic pathogens (Lamka et al., 1980; Geldreich et al., 1972), enhanced corrosion of pipe materials (Emde et al., 1992; Lee et al., 1980), and interference with the monitoring for indicator organisms because of high heterotrophic plate counts (HPCs) (LeChevallier & McFeters, 1985; Geldreich et al., 1972). Bacterial regrowth can also result in positive coliform samples (LeChevallier et al., 1987), which may either lead to false alarms regarding water quality or even mask true microbial contamination (LeChevallier, 1990). The factors that affect bacterial regrowth include temperature, amount of biodegradable organic matter (BDOM), disinfectant residual, corrosion products, and accumulation of sediments (LeChevallier et al., 1996; LeChevallier, 1990).

Biostability refers to the ability of water to support the growth of microorganisms. Biologically stable water does not promote the growth of bacteria, whereas biologically unstable water can support the growth of microorganisms to a significant extent (Rittmann & Snoeyink, 1984).

The potential of a water supply to support the growth of microorganisms can be quantified by measuring the amount of BDOM in the water. Two parameters frequently used to measure BDOM are assimilable organic carbon (AOC) and biodegradable dissolved organic carbon (BDOC) (Huck, 1990). To prevent or limit coliform regrowth in distribution systems containing a disinfectant residual, AOC concentration limits of 50 and 100 μg/L have been proposed (LeChevallier et al., 1992; LeChevallier et al., 1991).

However, the availability of BDOM as measured by AOC or BDOC is not the only important factor controlling regrowth (LeChevallier et al., 1996; LeChevallier, 1990). A recent study (Volk & LeChevallier, 2000) suggested that coliform regrowth is likely when the following three criteria are met: temperature at or above 15°C, disinfectant residual at or below 0.5 mg/L for dead-end free chlorine residuals or 1 mg/L for chloramine residuals, and plant effluent AOC concentrations above 100 μg/L.

The mechanisms by which microorganisms survive in distribution systems and how various water quality parameters affect their survival are still not fully understood. For many water utilities, the distribution system is a poorly understood “black box.” This study’s main goal was to improve understanding of the interplay between water quality and microbial regrowth in full-scale chloraminated distribution systems. The specific objectives were to evaluate the biostability and microbiological quality of the distribution system in the city of St. Paul, Minn., and to correlate operational and water quality parameters with the biological quality of the system.

MATERIALS AND METHODS

Treatment Plant and Study Site

The distribution system investigated was owned and operated by St. Paul Regional Water Services (SPRWS) in St. Paul. The city of St. Paul’s distribution system serves 343,000 customers over an area of 114 sq mi (295 km^2) using approximately 1,099 mi (1,768 km) of main. The mains are composed primarily of unlined cast iron.

Raw water is withdrawn from the Mississippi River, passed through a chain of lakes, and treated by a 50-mgd (189-ML/d) (annual average flow) treatment plant. During summer months, the raw water supply is often augmented with groundwater prior to treatment. Treatment processes include lime-softening, flocculation, sedimentation, disinfection, and filtration. The residual disinfectant is monochloramine. Typical water quality characteristics of the plant effluent for summer and winter are shown in Table 1.

Figure 1 is a map of the St. Paul distribution system area. Fifty sampling sites covering the entire distribution system were grouped into five regions based on their locations. The hydraulic residence times of these sites range from less than 1 hr to 19 days, as previously determined from a fluoride tracer study. Ten sites in the southwestern area of the distribution system (region 5) were sampled between July and December 1999. An additional sampling campaign encompassing the entire system was performed from January to April 2000. The full-scale distribution system sampling is summarized in Table 2. Further information about samples analyzed for AOC can be found in Tables 3 and 4.

Table 1 Monthly average water quality characteristics of the St. Paul Regional Water Services treatment plant effluent for July and November 1999

Water Quality	July 1999	November 1999
Temperature—°*C*	22	10
Turbidity—*ntu*	0.02	0.04
pH	9.0	8.9
Dissolved oxygen—*mg/L*	7.2	8.9
Total residual chlorine—*mg/L*	3.0	2.8
Dissolved organic carbon—*mg/L*	3.09	4.96
Total ammonia–nitrogen—*mg/L*	0.739	0.857
Nitrate, nitrite, nitrogen—*mg/L*	0.102	0.234
Chloride—*mg/L*	33.0	28.5
Total alkalinity—*mg/L* $CaCO_3$*	51	52
Total hardness—*mg/L* $CaCO_3$	79	86

*$CaCO_3$—calcium carbonate

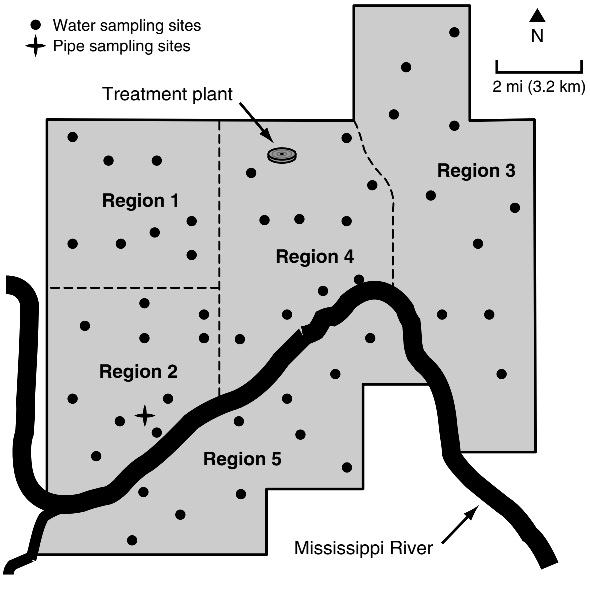

Figure 1 Map of the study area showing location of the sampling sites

Table 2 Distribution system sampling summary*

Sampling Location	Hydraulic Retention Time *days*	Sampling Period	Number of Samples	Number of Assimilable Organic Carbon Analyses
Plant effluent	0	July to December 1999	8	5
		January to April 2000	7	5
Region 1	< 1 to > 10	March 2000	9	1
Region 2	< 1 to > 10	January 2000	10	0
Region 3	< 2 to 14	February 2000	10	1
Region 4	< 1 to > 10	January to March 2000	20	1
Region 5	< 1 to 19	July to October 1999	16	11
		April 2000	10	2

* Water samples were analyzed for total chlorine, pH, temperature, dissolved oxygen, heterotrophic plate counts, coliforms, injured coliforms, total organic carbon, dissolved organic carbon, turbidity, and NH_3–N. Samples were also analyzed for NO_3^-–N beginning in January 2000.

Glassware Preparation

All of the glassware used in this study was prepared to be free of organic carbon by soaking it in 3 *N* (3 mol/L) HNO_3 for 24 hr, rinsing it with distilled water at least three times, then combusting it at 550°C for 6 hr to remove any remaining organic carbon.

Water Sample Collection

Water samples were collected in 250-mL organic carbon–free glass bottles and sterile 125-mL plastic bottles containing 0.01% (final concentration) sodium thiosulfate for dechlorination. The samples were collected from taps at the treatment plant and each sampling location after sterilizing the tap with flame and then flushing until the water reached a constant temperature, which indicated that the water was coming from the main. The samples were transported to the laboratory in coolers with ice packs and analyzed within 12 hr.

Table 3 AOC[*] concentrations at the plant effluent and at distribution system sampling sites when the water temperature was greater than 15°C[†]

Sampling Location	Sampling Period	Number of AOC Analyses	Temperature *°C*	Hydraulic Retention Time *days*	AOC Concentration *μg/L*	Mean AOC Concentration *μg/L*
Plant effluent[‡]	July to August 1999	4	21 to 22	0	117 to 167	148 ± 23[§]
Region 5[‡]	July to August 1999	7	21 to 23	1 to 19	117 to 227	153 ± 50[§]
Plant effluent[**]	October 1999	1	12	0	133	133
Region 5[**]	September to October 1999	4	17 to 23	11 to 13	151 to 194	163 ± 22[§]

[*] AOC—assimilable organic carbon

[†] Excluding the plant effluent sample in October 1999

[‡] Raw water supply consisted of surface water augmented with groundwater

[§] Mean value ± standard deviation

[**] Raw water supply consisted of surface water only

Table 4 AOC[*] concentrations at the plant effluent and at distribution system sampling sites when the water temperature was less than 15°C

Sampling Location	Sampling Period	Number of AOC Analyses	Temperature *°C*	Hydraulic Retention Time *days*	AOC Concentration *µg/L*	Mean AOC Concentration *µg/L*
Plant effluent[†]	February to April 2000	5	5 to 9	0	128 to 195	162 ± 24[‡]
Region 1	March 2000	1	8 > 10	156		
Region 2	NA[§]	0	NA	NA	NA	
Region 3	March 2000	1	8	19	169	164 ± 6[‡]
Region 4	March 2000	1	7	4	169	
Region 5	April 2000	2	9	11	168	
				<4	160	

* AOC—assimilable organic carbon

† Raw water supply consisted of surface water only

‡ Mean value ± standard deviation

§ NA—not applicable

Chemical and Physical Analyses

Total chlorine[*] (measured by the DPD colorimetric method), pH,[†] dissolved oxygen (DO),[‡] temperature,[†] and turbidity[§] were measured on-site using portable meters. Ammonia-nitrogen, nitrate-nitrogen, sulfate, and chloride were measured according to standard procedures (*Standard Methods*, 1995).

Microbiological Analyses

Total coliform bacteria were enumerated by the membrane filter procedure (*Standard Methods*, 1995) with 0.45-µm-pore-size membrane filters[**] using both m-Endo LES agar[††] and m-T7 agar.[††] HPC bacteria were enumerated by the spread plate procedure (*Standard Methods*, 1995) with R2A agar[††] incubated at room temperature (20–24°C) for seven days. Total bacteria (nonviable plus viable bacteria) were enumerated following an established procedure (Kepner & Pratt, 1994) with minor modification. Bacteria in the water samples were fixed by glutaraldehyde (final concentration 2.5%) and stained with 4,6-diamidino-2-phenylindole[‡‡] at a final concentration of 1 mg/L in the dark for 10 min. The stained bacteria were filtered onto a 25-mm, 0.2-µm-pore-size black polycarbonate membrane filter.[§§] The filters were then placed on a microscope glass slide, mounted with a drop of nonfluorescent immersion oil,[***] and covered with a clear glass cover slip. The slide was topped with another drop of immersion oil and mounted on an epifluorescence microscope[†††] with a magnification of 1,000. At least 20 fields per slide

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[*] LaMotte 1200, LaMotte Co., Chestertown, Md.

[†] Orion 230A, Thermo Orion, Beverly, Mass.

[‡] YSI 55, YSI, Yellow Springs, Ohio

[§] LaMotte 2020, LaMotte Co., Chestertown, Md.

[**] Type HA, Millipore Corp., Bedford, Mass.

[††] Difco Laboratories, Detroit, Mich.

[‡‡] Sigma Chemicals, St. Louis, Mo.

[§§] Type GT, Millipore Corp., Bedford, Mass.

[***] Citifluor, Ted Pella Inc., Redding, Calif.

[†††] Model Eclipse E600 with a 100W mercury lamp and a Nikon UC-2E/C 330–380-nm excitation filter, Nikon Corp., Tokyo, Japan

Total chlorine, pH, dissolved oxygen, temperature, and turbidity were measured on-site using portable meters.

were randomly selected and counted. Results were converted into number of bacteria per milliliter by multiplying by the conversion factor and the dilution factor. Total bacteria measurements were performed only on samples from the batch experiment.

Organic Carbon Analyses

Total organic carbon (TOC) was measured with an ultraviolet-persulfate organic carbon analyzer.* Dissolved organic carbon (DOC) was analyzed by the same method after the sample was filtered through precombusted (450°C for 6 hr) glass fiber filters† (nominal porosity 0.7 µm). AOC was measured following the rapid method (LeChevallier et al., 1993a), except that bacteria were enumerated using plate counts instead of adenosine triphosphate fluorescence. Yield values of 4.1×10^6 colony-forming units (cfu)/µg acetate-carbon and 1.2×10^7 cfu/µg acetate-carbon were used for P17 and NOX, respectively, to convert average plate counts from days 3, 4, and 5 of the incubation to AOC concentrations. Standard error values of the results from triplicate AOC measurements on the same water sample were less than 10%, indicating satisfactory reproducibility. The average AOC values obtained for sodium acetate standard solutions containing 100 µg/L acetate-carbon inoculated with P17 and NOX were 105 and 66 µg/L acetate-carbon, respectively.

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* Model Phoenix 8000, Tekmar Dohrmann, Cincinnati, Ohio

† Type GF/F, Whatman Inc., Maidstone, England

Pipe Sampling and Analyses

In July and August 1999, two sets of pipe samples were taken from the distribution system at a location with a hydraulic residence time of approximately four days. Three 8-in. (200-mm) long pipe sections were cut from a 6-in. (150-mm) diameter main at one location in July. Two 8-in. (200-mm) long pipe sections were cut from another nearby location in August. The pipe sections were capped at one end using presterilized (soaked in 100-mg/L chlorine solution for several hours) rubber caps with metal clamps, filled with dechlorinated water from the same main until overflowing, capped at the other end, placed in coolers with ice packs, and transported to the laboratory. Once in the laboratory, one cap was taken off the pipe section, and the water was poured out. After it was rinsed with autoclaved phosphate buffer, the interior of the pipe section was scraped with sterile spatulas, and the loosely bound solid material scraped from the interior pipe wall was collected into sterile containers. Part of the material was put into ceramic dishes to determine total solids and volatile solids. The remaining material was homogenized following an established procedure (Camper et al., 1985) and used to determine HPC density as described earlier. Finally, both the solids and HPC data were normalized by the corresponding interior surface area of the pipe sections.

Batch Experiment

A batch experiment was conducted to separate the individual role of monochloramine residual in bacterial regrowth from other factors such as the pipe walls and system hydraulics. Treatment plant effluent was collected Apr. 27, 2000, and placed into 160-mL serum bottles, which were grouped into three series.

Series 1 (the control series) was used without modification and thus served as the control. The water in series 2 (the dechlorinated series) was dechlorinated by adding 400 μL of a 13.2-g/L sodium thiosulfate solution to each bottle. The water in series 3 (the inoculated series) was dechlorinated in the same way and inoculated with bacteria by adding 100 μL of prefiltered lake water to each bottle to reach a final HPC bacteria density of approximately 10^3 cfu/mL. The lake water was prefiltered through glass fiber filters* (nominal porosity 2.7 μm) to remove large debris or algae. All of the serum bottles were incubated in

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* Type GF/D, Whatman Inc., Maidstone, England

the dark at room temperature (22–25°C) for 51 days. Periodically, three bottles from each series were sacrificed to measure water quality characteristics, including pH, DO, total chlorine, ammonia-nitrogen, TOC, and DOC. AOC concentrations were measured for each series after 0, 7, 21, and 51 days of incubation.

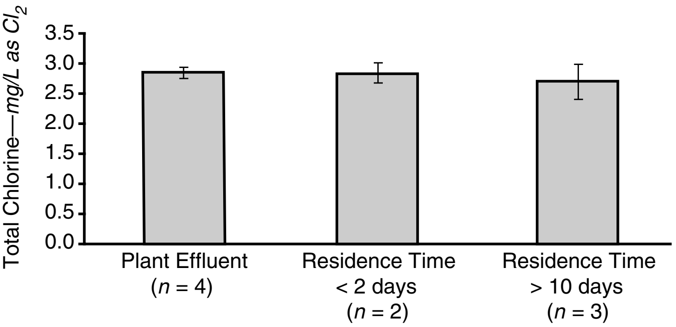
RESULTS AND DISCUSSION

Full-Scale Water Sampling

Figure 2 is a representative graph of the total chlorine concentration as a function of hydraulic retention time in the system. In contrast to some systems where chlorine residual was found to decrease substantially with retention time (e.g., Woolschlager et al., 1999; Chen & Weisel, 1998), little change in total chlorine concentration was observed between samples taken at the plant effluent, at sites with retention time less than two days, and at sites with retention time greater than 10 days.

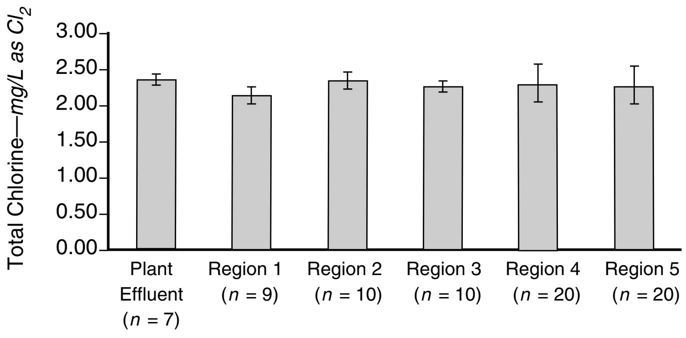
This trend was consistently observed during the full-scale study for all water quality parameters. Consequently, average total chlorine concentration was computed for the samples taken in each of the five regions and compared with the average plant effluent concentration (Figure 3).

In spite of the large variation of hydraulic retention time of the sampling sites in each region, the total chlorine concentration showed very little variation, as demonstrated by the small standard deviation (i.e., error bars in Figure 3). Although the data shown in Figure 3 were



Raw water supply comprises surface water augmented with groundwater; samples taken in region 5 in July and August 1999 when water temperature was >15°C. *n*—number of samples

Figure 2 Effects of retention time on total chlorine concentration



Raw water supply comprises surface water only; samples taken between January and April 2000 when water temperature was <15°C. *n*—number of samples

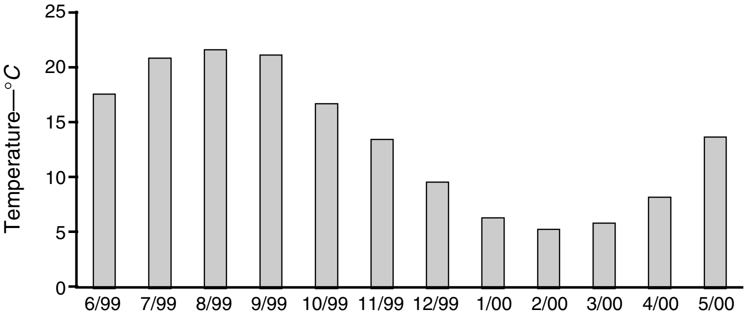
Figure 3 Average total chlorine concentration at plant effluent and in each sampling region

obtained between January and April 2000 when the water temperature was less than 15°C, sampling results in July and August 1999 (region 5 only), when the water temperature was greater than 15°C, showed the same trend (Figure 2).

During the sampling campaign, the average chlorine residual in the plant effluent was 2.36 mg/L. The average chlorine residual in the samples taken from the five regions ranged from 2.11 to 2.32 mg/L. Therefore, it can be concluded that there was little decrease of disinfectant residual in the distribution system. The limited chloramine decay was supported by the lack of accumulation of ammonia-nitrogen or nitrate-nitrogen (from oxidation of ammonia) in the system (data not shown). In addition, both DO and pH were stable throughout the distribution system.

The average water temperature in the distribution system from June 1999 to May 2000 is shown in Figure 4. For about two thirds of the year, the water temperature was below 15°C. Only in the summer months (July, August, and September) did the water temperature consistently exceed 15°C.

Tables 3 and 4 show the AOC concentrations at the plant effluent and at distribution system sampling sites when the water temperature was greater or less than 15°C, respectively. In both cases, there was little decrease of AOC in the distribution system. The slightly higher AOC concentration in the plant effluent when the water temperature was lower than 15°C was likely because of the fact that the raw water supply was not



Two hundred samples taken every month from the distribution system; standard deviation <2°C.

Figure 4 Average water temperature in the distribution system

augmented with groundwater, which is known to have a lower organic carbon content than surface water. The average AOC concentration of the plant effluent is greater than the AOC concentration recommended for limiting coliform regrowth in distribution systems maintaining a chlorine residual (50–100 µg/L) (LeChevallier et al., 1992; LeChevallier et al., 1991). However, AOC did not appear to be consumed in the distribution system. Similarly, little decrease in DOC was observed in the distribution system (data not shown).

In more than 84% of the distribution system samples (n = 70), the HPC density was too few to count (TFTC) (*Standard Methods*, 1995). No coliform bacteria (using m-Endo LES agar) or injured coliform bacteria (using m-T7 agar) were detected in either the plant effluent (n = 15) or the distribution system samples (n = 70).

In summary, little microbial activity was found in the system, as indicated by the stable AOC concentrations, low HPC density, and absence of coliforms in the water. Furthermore, the absence of nitrate production suggested that the system was free of nitrification problems. Nitrification is a two-step process during which ammonia is converted into nitrite, then to nitrate. Although nitrite was not monitored, any nitrite produced from ammonia oxidation would have been rapidly oxidized to nitrate given the high chlorine residuals in this system. Therefore, all of the data collected suggested that there was little or no bacterial regrowth in the system.

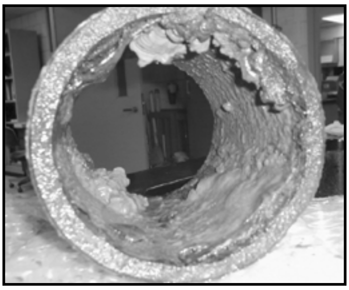
Full-Scale Pipe Sampling

The cast-iron pipe samples taken at adjacent locations in July and August 1999 had a significantly different appearance. The pipe samples taken in July 1999 had little corrosion on the interior pipe wall. There was a thin black coating on the pipe wall, which was easily removed by scraping with a spatula. The pipe samples taken in August 1999 had large orange tubercles on the pipe wall and soft black materials beneath the tubercles.

For the pipe samples with little corrosion (taken in July), the bacterial density on the pipe wall was 220 cfu/cm^2 ($n = 1$), and the ratio of volatile solids to total solids was 0.83 ($n = 1$). For the pipe samples with large tubercles (taken in August), the bacterial density on the pipe wall was 25 cfu/cm^2 ($n = 1$). The ratio of volatile solids to total solids was 0.12 for the tubercles and 0.15 ($n = 1$) for the soft black materials, respectively.

The materials recovered from the pipes without tubercles were high in relative organic content, suggested by the high ratio of volatile solids to total solids, whereas the materials recovered from pipes with large amounts of tubercles were high in inorganic content, as shown by the low ratio of volatile solids to total solids. Such results were expected because tubercles are primarily composed of deposits of ferric oxides/hydroxides. The materials recovered from the pipes without tubercles were believed to consist primarily of deposits of natural organic matter and possibly biofilms.

HPC densities on distribution system pipe surfaces reported in the literature range from 10 to 10^8 cfu/cm^2 (Woolschlager et al., 1999; Donlan & Pipes, 1988; LeChevallier et al., 1987). The bacterial densities recovered in this study were obviously at the low end of this range. Although the pipe-sampling campaign was extremely limited, the results



The pipe samples taken in August 1999 had large orange tubercles on the pipe wall and soft black materials beneath the tubercles.

suggested that there was little microbial colonization of the pipe walls, which is consistent with the findings from the full-scale system water sampling discussed earlier.

The pipe samples were taken from two locations approximately 150 m (492 ft) apart from each other on the same water main. The reasons for the different appearance between the two sets of pipe samples are not known. One possible explanation was that the pipe samples with little corrosion (July sampling) were taken at the upstream end of the main, closer to a 12-in. (300-mm) main where it is exposed to higher flow velocities. Higher flow velocities may have helped to limit the accumulation of corrosion products on the pipe wall (LeChevallier, 1990).

Batch Experiment

The objectives of the batch experiment were twofold: (1) to study chloramine decay and other water quality parameters over an extended time in a well-controlled system and (2) to examine the role of a monochloramine residual in maintaining the biostability and microbiological quality of the system.

The total chlorine concentration in the batch bottles during the 51-day incubation period is shown in Figure 5. For the dechlorinated series and inoculated series, chlorine residual was neutralized with thiosulfate, and the residual was zero. For the control series, the total chlorine concentration was initially at 2.46 mg/L and decreased to 1.82 mg/L after 14 days of incubation. Even after 51 days of incubation, a significant

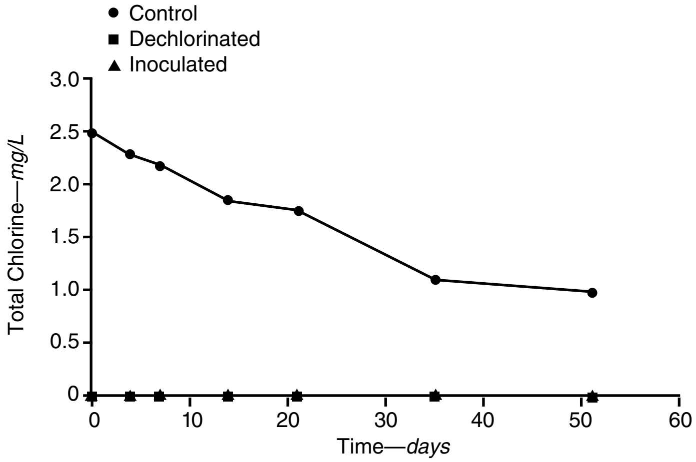


Figure 5 Total chlorine concentration versus time in the batch experiment

residual of 0.97 mg/L remained. The monochloramine decay was modeled using first-order and second-order kinetics, and the decay coefficients are given in Table 5.

The AOC results from the batch experiment are shown in Figure 6. The AOC concentration in the plant effluent sample collected for the batch experiment (210 µg/L) was higher than the values observed from the full-scale system sampling campaign (162 ± 24 µg/L) (Figure 5). This was likely a result of temporal changes in the raw water quality or AOC

Table 5 Kinetic parameters from the modeling of monochloramine decay in the batch experiment

Kinetics	Equation	R^2
First order	$-\frac{d[NH_2Cl]}{dt} = (0.0195\ \text{day}^{-1})[NH_2Cl]$	0.97
Second order	$-\frac{d[NH_2Cl]}{dt} = (0.0132[\text{mg/L}]^{-1}\text{day}^{-1})[NH_2Cl]^2$	0.96

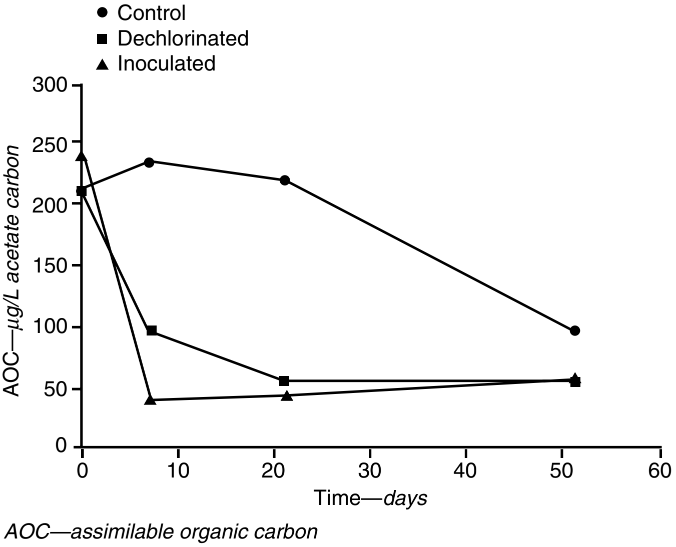


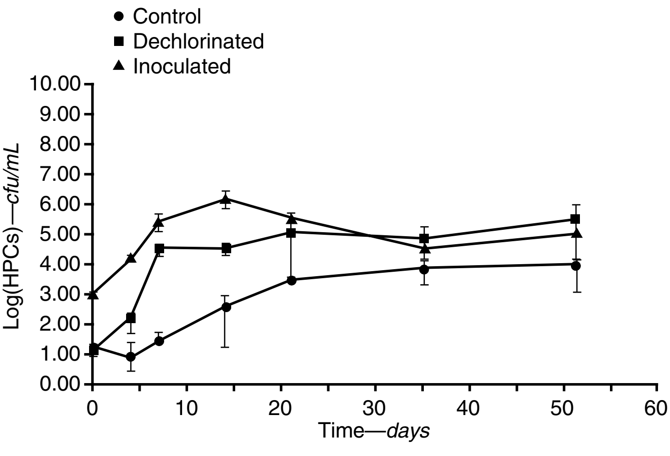
Figure 6 AOC concentration versus time in the batch experiment

removal efficiency in the treatment plant. Because of the addition of lake water, the initial AOC concentration in the inoculated series was 31 µg/L acetate-carbon higher than those of the other two series.

In all three series, a measurable decrease in AOC concentration was observed by the end of the 51-day incubation period; however, the AOC decrease followed a distinct pattern for each series (Figure 6). In the control series, in which no sodium thiosulfate or indigenous bacteria were added, the AOC concentration was relatively stable during the first 21 days of incubation. This result agrees with the lack of AOC degradation observed in the distribution system, where the maximum residence time is approximately 19 days. Between days 21 and 51, there was a large decrease in AOC concentration from 210 to 96 µg/L acetate-carbon. In the dechlorinated series, the AOC concentration decreased from 210 to 96 µg/L acetate-carbon after 7 days and to 54 µg/L acetate-carbon after 21 days of incubation. There was no further decrease in AOC concentration for the next 30 days. The AOC concentration in the inoculated series decreased rapidly to 40 µg/L acetate-carbon after 7 days of incubation, and there was no further decrease.

Unlike AOC, the DOC concentration in all three series did not change during the batch incubation period (data not shown). This is not surprising because AOC made up only a small fraction of the total DOC (< 4% in this case).

The HPC density in the three series also followed distinct patterns (Figure 7). In the control series, the HPC density did not begin to

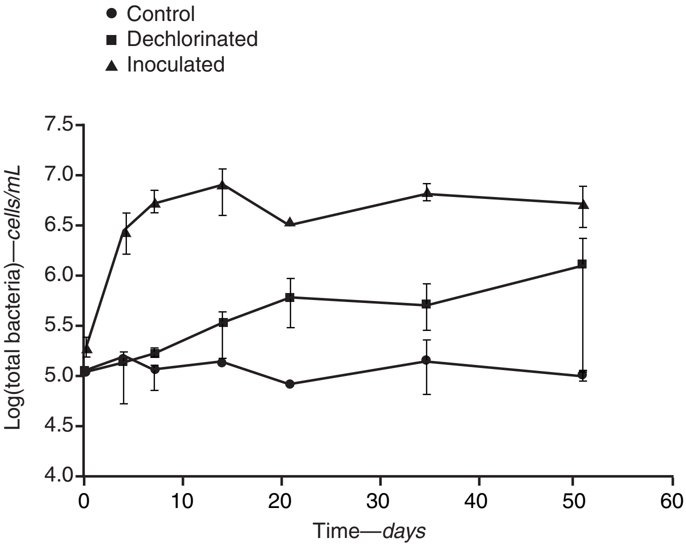


HPC—heterotrophic plate count; error bars represent range of data

Figure 7 HPC density versus time in the batch experiment increase until sometime between day 7 and day 21. From day 7 to day 21, there was a 2-log increase in HPC density. Then the HPC density increased relatively slowly, reaching $10^{4.02}$ cfu/mL by the end of incubation. In the dechlorinated series, the largest increase occurred in the first 7 days, during which there was a 3-log increase. Following that, the HPC density increased at a much slower rate and reached $10^{5.7}$ cfu/mL by the end of incubation. Similarly, in the inoculated series, the largest increase occurred in the first seven days. At the end of incubation, the HPC density was $10^{5.13}$ cfu/mL.

The change in total bacterial counts with time for the three series also differed, as shown in Figure 8. In the control series, there was little change throughout the incubation period, and the total bacterial count fluctuated around $10^{5.0}$ bacteria/mL. In the dechlorinated series, the total bacterial count began at $10^{5.0}$ bacteria/mL and gradually increased to $10^{5.7}$ bacteria/mL after 35 days of incubation. In the inoculated series, the total bacterial count began at $10^{5.3}$ bacteria/mL, increased rapidly during the first 7 days of incubation, reached a maximum of $10^{6.9}$ bacteria/mL, and then stabilized.

The observed increases in HPC density in the dechlorinated series and the inoculated series corresponded with the observed decreases in their AOC concentrations. A similar trend could be found between the



Error bars represent range of data.

Figure 8 Total bacterial density versus time in the batch experiment total bacterial counts and AOC concentration. However, a similar trend was not observed for the control series. First, although there was little decrease of AOC from day 0 to day 21, there was a 2-log increase in HPC density. A likely explanation is that the increase of HPC density is a result of recovery from chloramine injury or dormancy instead of regrowth (Gibbs et al., 1993; Jacangelo & Olivieri, 1985). Between day 7 and day 14, the increase in HPC density occurred at approximately the same time as the chlorine residual decreased below 2 mg/L. This suggests that a combined chlorine residual of 2 mg/L was required to suppress bacterial recovery from dormancy or injury. This hypothesis may help to explain the lack of microbial activity in the full-scale system where the combined chlorine residuals are consistently above 2 mg/L. Second, although the AOC concentration decreased more than 100 µg/L acetate-carbon between day 21 and day 51, there was only a 0.5-log increase in HPC density. The presence of disinfectant in the control bottles likely limited the net increase in HPC bacteria density in comparison with the other two series without chlorine.

It is difficult to compare the increase in HPC bacteria per unit decrease in AOC concentration (i.e., yield values) among the three series for two reasons. First, the microbial population in the inoculated series is likely different from that in the other two series because of the addition of lake water. Different microbial populations would be expected to exhibit a different yield in response to utilization of the same AOC materials. Second, any observed change in HPC density in the control bottles was a net result of increase from AOC consumption and decrease as a result of disinfection.

For the control series, the presence of a chloramine residual > 2 mg/L inhibited microbial activity for the first 21 days of the batch experiment; however, removal of the residual chlorine resulted in rapid bacterial regrowth, as shown by the AOC, HPC, and total bacterial count results from the dechlorinated series. The AOC results of the dechlorinated series also showed that bacteria that are capable of utilizing the easily assimilable organic matter are present in the treated water even though few HPC bacteria were recovered from the plant effluent and the plant effluent had a sustained monochloramine residual of 2.4 mg/L or greater. The inoculated series was used to evaluate the effect of adding a significant dose of active heterotrophic bacteria on the AOC utilization rate. The results suggested that the addition of indigenous bacteria increased the rate but not the extent of AOC utilization compared with the dechlorinated series.

REASONS FOR SYSTEM BIOSTABILITY

The results from the full-scale monitoring, batch experiment, and pipe sampling showed that the water in the SPRWS distribution system was biologically stable and that there was no significant microbial activity in the system. This result was somewhat surprising, given the moderate to high AOC concentration in the treated water (162 ± 24 µg/L). With reference to a recent study (Volk & LeChevallier, 2000), the distribution system in this study consistently violated one of the threshold criteria (AOC) and violated two of the three threshold criteria in the summer (temperature and effluent AOC). However, no regrowth of coliform or HPC bacteria was detected. This observation is attributed to the following factors.

The Treatment Plant Uses Monochloramine as the Terminal Disinfectant

The growth of bacteria was studied in both biofilms and bulk water using a pilot reactor system to simulate drinking water distribution systems (van der Wende et al., 1989). It was concluded that biofilm growth and detachment accounted for most, if not all, of the planktonic cells in the bulk water, regardless of whether a disinfectant residual was present. Therefore, the key to controlling the growth of bacteria in distribution systems appears to be controlling the growth of biofilms in the system.

It was reported that monochloramine is more effective than free chlorine at inactivation of biofilm bacteria (LeChevallier et al., 1988a), as greater penetrating power of monochloramine into biofilms compensates for its limited disinfection activity. Another study indicated that disinfection by free chlorine was negatively affected by microbial attachment to surfaces, whereas disinfection by monochloramine was only somewhat affected by attachment to surfaces (LeChevallier et al., 1988b).

In addition, monochloramine is typically less corrosive and more stable than free chlorine (LeChevallier et al., 1990), and disinfection by monochloramine was shown to be less affected by corrosion than free chlorine (LeChevallier et al., 1993b). A full-scale study in Canada compared the effects of chlorine and chloramine on bacterial regrowth and concluded that chloramine was more effective as a secondary disinfectant (Neden et al., 1992). Therefore, in terms of controlling bacterial regrowth in distribution systems, monochloramine appears to be more effective than free chlorine.

A High Residual Chlorine Was Successfully Maintained Throughout the Distribution System

In the St. Paul distribution system, even at locations with hydraulic residence times of more than 10 days, the residual chlorine was still 2.1 mg/L or higher. It was suggested that a 2-mg/L monochloramine residual was able to inactivate biofilm bacteria on iron pipes (LeChevallier et al., 1990). Although the threshold level is likely to vary with water quality and pipe characteristics, a similar threshold value was observed in this study.

Water Temperature Is Below 15°C for Much of the Year

Research has shown that bacterial regrowth (LeChevallier et al., 1991) and nitrification (Wilczak et al., 1996) problems are associated with water temperatures of 15°C or higher. The water temperature in the St. Paul distribution system was less than 15°C for 70% of the year, which likely helped to limit bacterial regrowth.

The City of St. Paul Water Utility Has a Good Maintenance Program for the Distribution System

Since August 1999, the treatment plant has been adding corrosion inhibitor (orthophosphate and polyphosphate) to the finished water. Because corrosion interferes with disinfection of biofilms (LeChevallier et al., 1993b) and tubercles protect bacteria from disinfectants (LeChevallier et al., 1987), controlling corrosion by adding corrosion inhibitor should aid in reducing the potential for regrowth. The city of St. Paul water utility also implemented a regular distribution system flushing routine. It was reported that all solid materials recovered from the distribution system, including floc material and sediment, had high densities of HPC bacteria (LeChevallier et al., 1987). Similarly, another study showed that the majority of viable bacteria in chlorinated drinking water were attached to particles (Ridgway & Olson, 1982). Therefore, removing particulate materials from the distribution system periodically by flushing probably helped to limit bacterial regrowth in the system.

CONCLUSIONS

On the basis of the results from the full-scale distribution system monitoring and the batch experiment, the authors offer the following conclusions:

- The water in the city of St. Paul water distribution system was biologically stable, and there was no significant microbial activity in the system.
- The excellent biostability and microbiological quality of the system were attributed to a combination of factors, including use of monochloramine as the terminal disinfectant, a high residual chlorine in the system (greater than 2 mg/L), and low water temperature (less than 15°C) for much of the year.
- Maintenance of a significant monochloramine residual (greater than or equal to 2 mg/L) is imperative for controlling regrowth because bacteria capable of utilizing AOC materials were present in the treated water and have likely colonized the distribution system.
- Effluent AOC concentration may not be the dominant factor for controlling bacterial regrowth in all distribution systems. Biostability can be achieved in a water supply that contains moderate to high AOC concentrations by maintaining a high disinfectant residual throughout the distribution system. However, rapid bacterial growth will likely occur if chloramine residuals are depleted.

ACKNOWLEDGMENTS

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CHAPTER · 5

Developing a Simple Factor to Evaluate Microbiological Stability in Distribution Systems

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BACKGROUND

It is well established that water quality deteriorates in distribution systems. Two major water quality issues are the loss of disinfectants and the growth of bacteria. Since utilities are charged with protecting public health, they must limit biological activity in drinking water through maintaining an effective disinfectant to the consumer's tap. To achieve effective disinfection, the disinfectant rate must exceed the bacterial growth rate. This research proposes a simple method to establish effective disinfection by using a "stability point" equation that compares the disinfection rate to the bacterial growth rate by applying fundamental biological modeling concepts and parameters derived from batch tests.

INTRODUCTION

The most prevalent type of microorganisms found in distribution systems is heterotrophic bacteria [1–3]. Heterotrophs use biodegradable organic matter (BOM) as an energy and carbon source. BOM typically exists in significant concentrations in source waters, especially surface waters, and is not removed by traditional water treatment processes [4]. Because of the abundance of their substrates, heterotrophic plate counts are found in concentrations of up to 106 counts per milliliter in distribution system water samples [1–3].

Bacteria in distribution systems exist as cells suspended in the bulk water or cells fixed on pipe surfaces. Because modeling suspended-bacterial processes is generally simpler than for fixed-bacterial processes, this chapter focuses on suspended bacteria [5]. Consideration of bacteria attached to pipe surfaces is dealt with elsewhere [6].

This chapter develops a simple expression that can be used to estimate bacterial stability within distribution systems. Experimental data gathered for this study resulted from a yearlong sampling program conducted at the United Water drinking water distribution system located in the northeast corner of the state of New Jersey, USA, documented elsewhere [7].

BACTERIAL GROWTH AND DECAY

The mathematical relationship chosen as the basis for modeling bacterial synthesis was originally developed by Monod [8] and adapted for engineering applications by Lawrence and McCarty [9]:

$$r_{syn} = Yq_m = \left[\frac{S}{K+S}\right]X \tag{1}$$

where

r_{syn} = rate of bacterial synthesis [$M_X\ L^{-3}\ T^{-1}$]
Y = biomass yield factor [$M_X\ M_S^{-1}$]
q_m = maximum specific substrate utilization rate [$M_S\ M_X^{-1}\ T^{-1}$]
K = half-maximum rate concentration [$M_S\ L^{-3}$]
S = limiting substrate concentration (BOM for heterotrophs) [$M_S\ L^{-3}$]
X = concentration of active biomass [$M_X\ L^{-3}$]

(Note that generic units of mass [M], length [L], and time [T] are used for the equations presented in this chapter.)

Another factor that affects the bacterial concentrations is endogenous decay, which is the self-oxidation of active biomass to satisfy the cells' maintenance energy demands, such as motility, osmotic regulation, and heat loss. For slow-growing bacteria typical of oligotrophic aquatic environments, endogenous decay is a major factor that influences the net accumulation of biomass, because decay rates can approach a magnitude similar to synthesis rates. Endogenous decay can be expressed by:

$$r_{dec} = -bX \tag{2}$$

where

r_{dec} = rate of biomass loss due to decay $[M_x L^{-3} T^{-1}]$
b = endogenous decay rate coefficient $[T^{-1}]$

The endogenous decay coefficient is constant regardless of the bacterial synthesis stage [10].

DISINFECTION

Chlorine is commonly used as a secondary disinfectant in distribution systems. However, chlorine residuals do not eliminate bacteria in distribution systems. In fact, several studies, including this one, have found high bacterial counts exceeding 10^6 per mL in the presence of significant concentrations of chlorine [1–3, 6]. Since chlorine is not "sterilizing" distribution systems, the interplay of disinfection with other factors must be considered.

An early model of disinfection, shown by Equation 3, was developed by Chick [11] and Watson [12]:

$$r_d = -k_d[Cl_2]^n X \tag{3}$$

where

r_d = disinfection rate $[M_x L^{-3} T^{-1}]$
X = bacteria concentration $[M_x L^{-3}]$
Cl_2 = disinfectant concentration $[M L^{-3}]$
k_d = deactivation coefficient $[L^3 M^{-1} T^{-1}]$
n = coefficient of dilution (typically 1.0)

Although the Chick-Watson relationship was developed over 90 years ago, it is still used widely. One reason for its continued use is that it fits data from many types of disinfection studies. Because more complex disinfection models do not achieve a better fit to long-term disinfection in drinking water systems [13], the Chick-Watson model was used in this research.

DEVELOPING THE STABILITY POINT

The important question examined by this research is why do we find high bacterial counts in distribution systems? The major factors that control bacterial populations include substrate and disinfection concentrations. However, measured data trends cannot be used alone to sort out the specific contribution of each of these factors. To help sort these factors out, a simple mathematical expression was developed for what takes place when water is biologically stable. In the presence of disinfectants, biological stability is achieved when the disinfection rate equals or exceeds the net synthesis rate of bacteria. The point at which the disinfection rate just equals the net synthesis rate is defined here as the stability point. Using variables defined earlier, Equation 4 is the mathematical representation of the stability point:

$$k_d[Cl_2] = Y\left(q_m\left[\frac{S}{K+S}\right]\right) - b \qquad (4)$$

$$(\mathit{Disinfection} = \mathit{synthesis} - \mathit{endogenous\ decay})$$

DETERMINING PARAMETERS

To calculate the stability point of a particular water, the parameters used in Equation 4 must be determined (i.e., Y, q_m, K, b, k_d). Some parameters are relatively constant, such as the bacterial yield rate ($Y \approx 0.5$ µg COD cell/µg COD) and endogenous decay ($b \approx 0.1$ day^{-1}) [6, 9, 10, 17]. Derivation of the remaining parameters used to calculate the stability point involves two consecutive steps: (1) set biodegradable organic matter degradation parameters (q_m, K) from the biodegradable dissolved organic carbon (BDOC) tests, and (2) set disinfection parameter (k_d) from batch tests of treatment plant effluent after disinfection. Details of each of these steps are given in the following paragraphs.

Setting Biodegradable Organic Matter Parameters

Natural organic matter varies considerably in structure and functional group content. Because of these variations, the parameters that define the utilization of BOM by heterotrophic bacteria are location specific [14]. Parameters that define the heterotrophic utilization include the maximum specific substrate utilization rate (q_m) and the half-maximum substrate concentration (K), as described in Equation 1. In this study, biodegradable dissolved organic carbon (BDOC) tests were performed to determine the amount of BOM in water samples [15]. These tests involve mixing a water sample with bacteria in a batch reactor. The dissolved organic carbon (DOC) is measured over time, and the change in DOC is traditionally interpreted as the BOM concentration. However, the change in DOC is not equal to the true BOM because of the influence of soluble microbial products [16]. The Biodegradable Organic Matter (BOM) model was used to estimate the true amount of BOM from BDOC test results [17]. This model estimates the BOM concentration and degradation parameters by fitting the model to experimental results for DOC concentration over time. The BOM model divides DOC into three components: rapidly degraded (BOM1), slowly degraded (BOM2), and refractory organic matter (ROM). Figure 1 shows the results of one BDOC test and the best fit using the BOM model having a rapidly degraded fraction (BOM1) and a slowly degraded fraction (BOM2). Parameter values derived from this test are shown in Figure 1. The

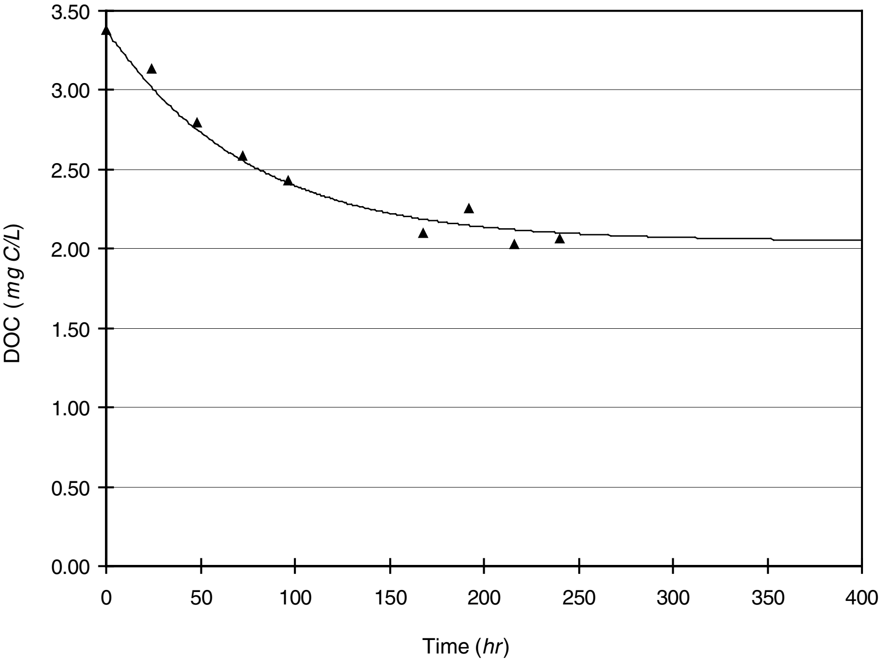

Figure 1 Use of the BOM model to obtain degradation kinetic parameters from BDOC tests. BDOC test of treatment plant sample. Percentages of BOM1, BOM2, and ROM are 10%, 45%, and 45%, respectively.

percentages of BOM1, BOM2, and ROM were 10, 45, and 45, respectively, for this sample.

Determining Disinfection Parameters

Once the BOM parameters are set, it is possible to determine the disinfection parameters. The batch test involves placing water samples in batch reactors and measuring the following species over time: chlorine (total and free), heterotrophic plate counts (HPC), and dissolved organic carbon (DOC). The parameters set in the batch test include the disinfection rate constants (k_d). The disinfection rate constant must be set in the batch test because the literature yields inconsistent values due to water and microbe-specific conditions. Figure 2 shows the results of a batch test and the fit to the results used to derive the disinfection parameters.

The disinfection rate was derived from the data in Figure 2 by accounting for heterotrophic net synthesis occurring simultaneously with

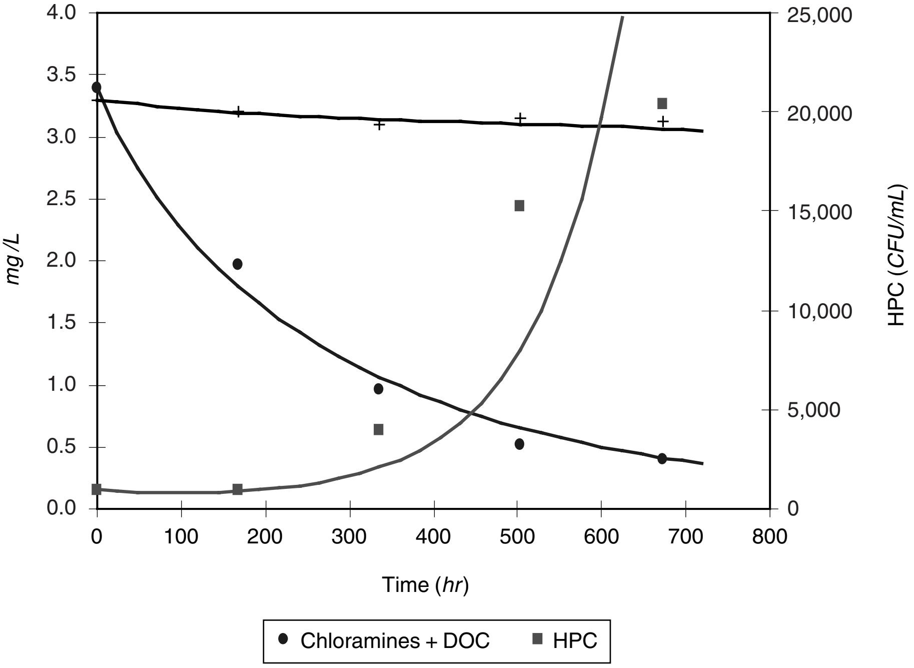

Figure 2 Batch test results used to determine the disinfection rate constants. Lines are the best fit to the batch test data using Equations 1, 2, and 3 and the parameters reported in this chapter.

disinfection. Heterotrophic net synthesis depends on substrate utilization and endogenous decay (Equations 1 and 2). The parameters that define substrate utilization were derived from the BDOC tests, as previously described. Because the net synthesis component was predictable, the disinfection rate could be set by fitting to the measured chloramine and heterotrophic counts from the batch test results using Equation 3, resulting in a value of k_d = 600 L-mole/hr.

RESULTS AND DISCUSSION

As discussed earlier, the main substrate (*S* in Equation 4) that supports heterotrophic synthesis in distribution systems is biodegradable organic matter (BOM). BOM can be divided into two fractions: the rapidly degraded fraction (BOM1) and the slowly degraded fraction (BOM2). Equation 4 is expressed graphically for each of these fractions of BOM as shown in Figure 3.

The left side of Equation 4, representing disinfection, is graphed as horizontal lines for several discrete concentrations of disinfectant representative of distribution system samples. The right side of Equation 4, representing net synthesis, is graphed as rising lines for increasing concentrations of BOM1 and BOM2 using degradation kinetics determined in the batch tests without disinfectant added. Points where disinfection and net synthesis lines cross are stability points.

From the BDOC tests, the average concentration of BOM2 in the treatment plant effluent is 1.5 mg/L as C. For BOM2, Figure 3 demonstrates that the stability point for BOM2 is less than 0.5 mg/L as Cl_2. However, the average concentration of BOM1 leaving the treatment plant was determined to be around 0.32 mg/L as C, making the stability point in the distribution system occur at chloramine concentration around 2.0 mg/L as Cl_2. The slope of the BOM1 curve is relatively steep. A decrease in BOM1 from 0.32 to 0.16 mg/L as C would bring the stability point near zero. On the other hand, increasing the BOM1 concentration to 0.80 mg/L as C would drive the stability point to a chloramine concentration of 4.0 mg/L as Cl_2, the maximum allowable disinfectant concentration. The key interpretation is that distribution system stability depends strongly on the BOM concentration, especially the rapidly degraded fraction (BOM1). This is not a new realization, because the importance of BOM in distribution system bacterial growth is well established [18–22]. What is new is the quantification of the relationship between BOM and disinfectant residual through the stability

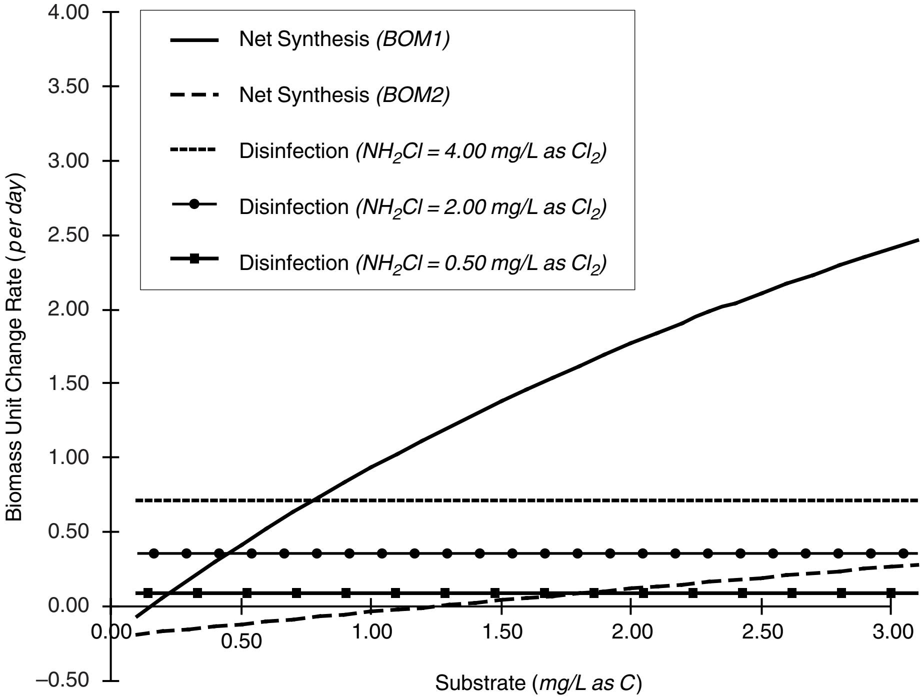

Figure 3 Biological stability points determined by comparing specific rates of disinfection and net synthesis for the two categories of biodegradable organic matter, BOM1 and BOM2. The stability point depends most strongly on the concentration of the rapidly degraded BOM1.

point. The quantification shows that small increases in BOM require large increases in disinfectant to maintain stability.

As noted earlier, this chapter focuses on the growth of suspended bacteria. Typically, bacteria in biofilms have lower disinfection rates than suspended bacteria. Therefore, to examine stability of biofilm growth using a simple tool such as the stability point, the disinfection rate constant must be adjusted lower. Our field study showed the rate of biofilm disinfection to be about half that of suspended cells [6]. However, the relationship of suspended and biofilm disinfection depends on the disinfectant type and the specific distribution system conditions. Water quality models have been developed to account for these phenomena [6, 23, 24].

CONCLUSIONS

The stability point is a useful tool to help utilities determine whether or not a combination of BOM and disinfectant concentrations can result in a biologically stable water in their distribution systems. Batch BDOC and disinfection tests provide bacterial growth and disinfection parameters needed for the stability-point equation. Plots of disinfection and bacterial growth rates, as shown by Figure 3, can help a utility determine what strategy can lead to biological stability, either by increasing disinfectant, decreasing organic growth substrate, or a combination.

ACKNOWLEDGMENTS

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CHAPTER · 6

Managing Water Quality in Consecutive Systems—The City of Coquitlam/GVRD Experience

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BACKGROUND

The city of Coquitlam is one of 17 member municipalities supplied with drinking water by the Greater Vancouver Regional District (GVRD). Changes to the regional water system over the last several years include a new ozone treatment facility, corrosion control, rechlorination, and other distribution system improvements. Of significance are key distribution system issues related to chlorine residuals, bacterial regrowth, and disinfection by-products.

In September 2000, the city of Coquitlam issued a boil water advisory within the Westwood Plateau community in response to excessive regrowth. The cause of regrowth during this period could not be attributed to one particular source, and it was determined that a combination of conditions provided an environment that was conducive to bacterial regrowth. The cooperative working relationship between the city and the GVRD is highlighted through the management of this event and other water quality issues.

INTRODUCTION

In the province of British Columbia, water supply systems are regulated under the Health Act, and the responsibility for enforcing the regulation lies with the medical health officer in each region. Under the regulation, a water purveyor is responsible for supplying "potable" water to its customers. The purveyors are responsible for monitoring the water supply as directed by the health officer and reporting the results to both the health officer and customers.

In the Greater Vancouver Regional District (GVRD) the district acts as a wholesaler of treated bulk water to its member municipalities. The municipalities in turn retail that water to their constituents. Both entities are considered purveyors and, therefore, both have to meet the obligations contained in the Health Act and its regulations. The GVRD is the sole supplier to the city of Coquitlam and most member municipalities and, therefore, assumes responsibilities for source treatment and for maintenance and monitoring of water quality to the point of delivery. The GVRD determines the measure of that quality in consultation with the member municipalities, which are responsible for maintaining and monitoring quality throughout their own distribution systems. Member municipalities provide dedicated sampling points, and GVRD staff collect the samples weekly. The GVRD water quality laboratory processes the samples and reports the results to both the member municipalities and their respective health authorities.

The city of Coquitlam and the GVRD were working cooperatively on water quality issues in the Westwood Plateau area prior to the boil water advisory issued in the fall of 2000. Monitoring on the plateau had shown that there were consistently low chlorine residuals in much of the system, and regrowth as measured by high HPC counts was occurring almost year-round. In the fall of 2000, the Summit Reservoir in Westwood Plateau started testing positive for total coliform. After two nonconsecutive overgrown coliform tests from the reservoir, the city was directed by the regional health officer to issue a boil water advisory to the residents receiving water from the Summit Reservoir. The city immediately sought the help and cooperation of the GVRD and requested:

- laboratory services for the testing of numerous water samples requested by the health authorities;
- an increase to the level of secondary disinfectant added at the Coquitlam Lake source; and

- advice to both the city and the health authorities on the event and interpretation of the sampling results.

The subsequent text highlights historical water quality conditions, emphasizing chlorine residuals, disinfection by-products, and HPC bacteria regrowth within the GVRD water supply system, and outlines how the city of Coquitlam and GVRD work together to resolve water quality issues such as the fall 2000 event in Westwood Plateau.

GVRD WATER SYSTEM

Source Water

Greater Vancouver, located in the southwest corner of the province of British Columbia, is the third largest metropolitan area in Canada. With a few exceptions, the water delivery system to the area is managed and controlled by the GVRD, which supplies bulk water at cost to its 17 member municipalities. The municipalities distribute the water to their consumers through municipal distribution systems. The average annual consumption is 1,100 ML/day, with peak daily flow of 1,500 ML/day in 2001.

The GVRD has three large impounding reservoirs located in the mountains in the northern area of the region and owns or controls the watershed lands that are defined by the catchment area of the supply lakes. Activities in the watersheds are limited to those related to the maintenance or improvement of the water supply. These protected watersheds have no industry, agriculture, transportation routes, or other factors that potentially could introduce pollutants associated with human activity. Thus, the source water is influenced only by naturally occurring factors. This untreated water is low in dissolved minerals and nutrients. The low mineral content results in water that is extremely low in alkalinity and hardness, such that the delivered water can be aggressive to metals and other materials used to construct the delivery system. The low nutrient level, particularly phosphates, is important because it severely restricts the amount of algal growth in the impounding reservoirs.

Prior to 1998, water treatment consisted only of coarse screening and the addition of chlorine for disinfection at the source. Chlorine added as a gas further reduces the alkalinity and the pH of the delivered water. Table 1 summarizes some of the key water quality parameters of the impounded water known as Coquitlam Lake. This lake supplies several member municipalities within the GVRD, including the city of Coquitlam.

Table 1 Coquitlam Lake source water (typical)

Parameter	Range	Average
Alkalinity, mg/L as $CaCO_3$	1.2–2.5	1.7
Hardness, mg/L as $CaCO_3$	2.6–3.1	2.7
TOC, mg/L	0.4–2.5	1.7
Turbidity (NTU)	0.17–2.1	0.42
Total Solids, mg/L	11–13	12
Total Dissolved Solids, mg/L	10–12	11
pH	6.1–6.5	6.2
True Color (TCU)	6–18	12

Historical Perspective

Given the abundant supply of “unpolluted” source water, it was generally believed that the local water was pristine and that the addition of minimal amounts of chlorine at the source was sufficient water treatment. It was also believed that occasional episodes of high turbidity, although a nuisance and unsightly, were not a hazard to public health. Green staining of plumbing fixtures resulting from corrosion of copper plumbing was simply an inconvenience. There were few coliform bacteria in the source water, and none could be measured in samples immediately following disinfection, so there was little need to monitor further out in the municipal systems. With no sewage contamination from either humans or agriculture, viruses and parasites were of no concern. The GVRD monitored the water in the large transmission system for coliform bacteria, but there was no great need to monitor in the municipal systems.

This perception changed in 1984. Monitoring began in one of the municipalities that detected the presence of total coliform in its system. The organisms appeared in the late summer/early fall, lasted for a few months, then disappeared. The same phenomenon was found when monitoring was expanded into other municipal systems. Coliforms were present throughout the municipal systems, particularly in areas with low water consumption. The detected organisms were total coliforms. Fecal coliform or *E. coli* was not detected.

As a result of the concern with the presence of coliforms and changing public perceptions and expectations, other water quality issues were raised. A major program to improve the quality of the water supplied to the consumer was initiated. Four main issues needed to be addressed: seasonal presence of coliform bacteria in the distribution system, periodic excessive turbidity in the source water, corrosiveness of the water, and inadequate primary disinfection to inactivate parasites.

Distribution System Improvements

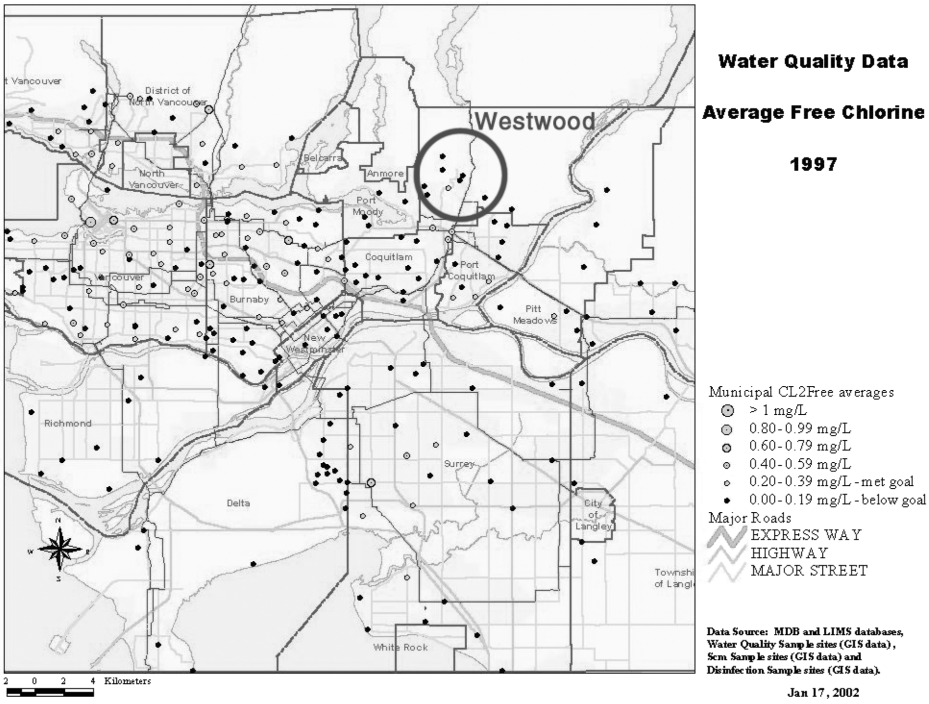
Water quality improvements in the past decade have included numerous facilities and operating procedures such as upgraded primary disinfection facilities that now consistently apply chlorine or ozone. These changes resulted in higher chlorine levels entering the transmission system with the aid of several secondary disinfection stations, which reapply chlorine to increase levels before the water enters the municipal systems. Better operation of the municipal systems and improved control of turbidity entering the transmission system have also helped to maintain water quality in the distribution piping.

The presence of coliform bacteria was believed to be the result of bacterial regrowth. The organisms were not detected in the large regional transmission mains, but were normally detected throughout municipal distribution systems, predominantly in low-flow areas. The occurrences coincided with the warmest water temperatures and the lowest chlorine residuals in the delivered water. The short-term solution to the regrowth problem was to construct facilities that would ensure an adequate chlorine residual in the municipal distribution systems.

The first stage of the improvement initiative involved increasing the level of chlorination at some existing facilities and the construction of new facilities. In practical terms, the goal was to increase the chlorine in GVRD mains to a minimum of 0.5 mg/L. This would ensure that all the water delivered to member municipalities would have a free chlorine residual of between 0.5 and 1.0 mg/L, the level believed to be required to achieve a minimum chlorine residual of 0.2 mg/L in most of the municipal distribution systems.

By mid-1999, eight large regional rechlorination stations were operational and now help to maintain chlorine levels in the GVRD transmission and distribution systems of member municipalities. Figures 1 and 2 highlight the improvements made by construction of the rechlorination stations.

In addition to rechlorination, other initiatives to decrease levels of bacteria were undertaken and included improved circulation of the water in storage reservoirs, flushing and cleaning of mains in the municipal systems, and operating the supply system to minimize low-flow areas.



Data Source: MDB and LIMS databases, Water Quality Sample sites (GIS data), Scm Sample sites (GIS data) and Disinfection Sample sites (GIS data).

Jan 17, 2002

Figure 1 1997 free chlorine

The impact of actions taken to date is quite significant; however, there are still problems with coliform occurrence in municipal systems, especially in the late summer and early fall periods. The coliform occurrence has been attributed to regrowth of organisms within the biofilm on the walls of the piping system and their subsequent release into the water stream. The coliforms are not thought to result from breakthrough of primary disinfection from the source or from intrusion into the piping system from outside the distribution system.

At the time, the GVRD laboratory was carrying out HPC or standard plate counts on all samples of water. These HPC counts did not accurately reflect the results of the coliform analysis as there was no noticeable increase during the period in which the coliforms were present. The HPC counts were being determined by the pour plate method using Typtone Glucose Extract Agar incubated at 35°C, referred to hereafter as the pour plate HPC. Consultants hired by the GVRD suggested that an alternative method would be more useful in measuring bacterial regrowth, a method that selected aerobic bacteria that would

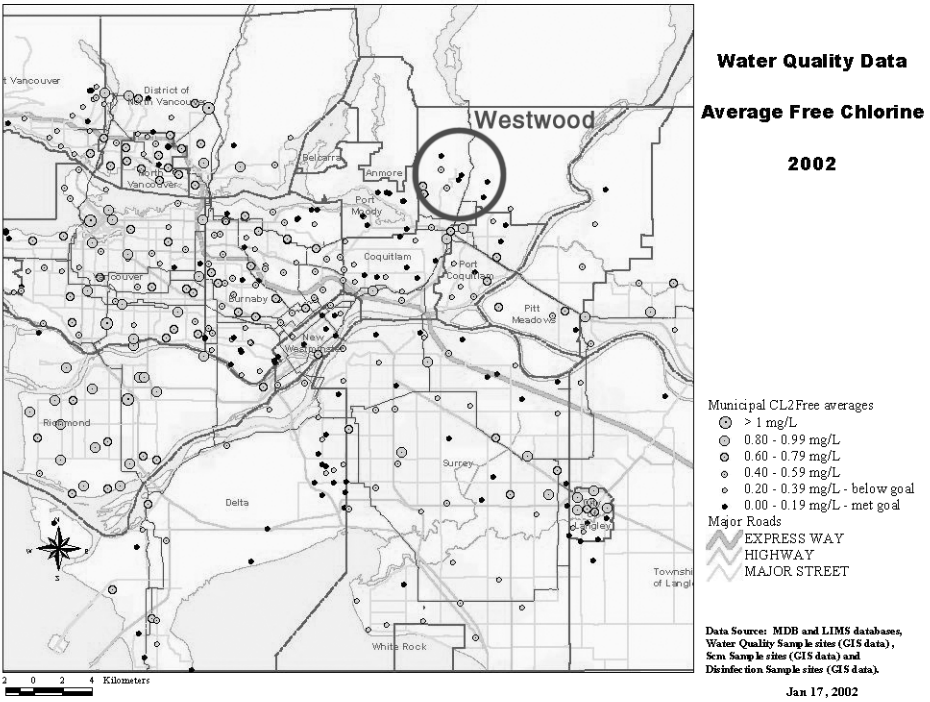


Figure 2 2002 free chlorine

grow slowly at relatively low temperatures on a low-nutrient agar. The medium selected was R2A with incubation for five days at 28°C (spread plate HPC).

A comparison between the two methods showed that the test method used had a significant effect on the number of bacteria detected. The spread plate method was detecting numbers three to four orders of magnitude higher than the pour plate method. The initial test was carried out during a regrowth period so it was decided to switch to the spread plate method for a year to evaluate the results over a wide variety of conditions.

The results from the initial year of monitoring were significant. When water temperatures were below 6°C, the HPC counts were low in samples collected from water mains. When the water temperature rose above 6°C, the HPC counts increased at those sites where there was little or no chlorine residual. These were the same areas that tested positive for coliforms, but the coliforms were not detected until the water temperature rose above 14°C. Figure 3 is an example of typical results for HPC

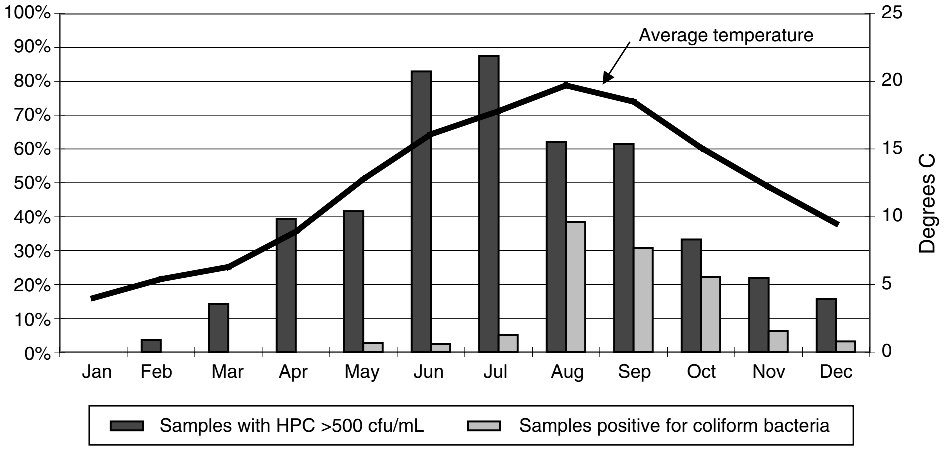


Figure 3 Typical low-flow regrowth area (1997)

and coliform analysis from water in low-flow areas in a member municipality known as Delta Control Area 1997.

The latest version of the Guidelines for Canadian Drinking Water Quality recommends that HPC bacteria be maintained below 500 cfu/mL. Prior to the construction of the rechlorination facilities, HPC counts throughout the municipal distribution systems were unacceptably high, as shown in Figure 4. Monitoring for HPCs as well as coliforms allowed the GVRD to monitor the effectiveness of the increased chlorine throughout the year instead of just the few months when coliforms were present.

With a few notable exceptions, the HPC bacteria are now at acceptable levels throughout the supply system and the coliform levels in most areas now comply with the BC Safe Drinking Water Regulation (1992). When the HPC levels begin to increase, regrowth is beginning and chlorination levels can be adjusted accordingly. The regrowth issues in the GVRD supply mains have been addressed, but there were still concerns with water quality in finished water reservoirs. All water has a chlorine demand; therefore, a chlorine residual will dissipate in a relatively short period of time (36 hours for GVRD water). Unless the retention time in a reservoir containing the water supplied by the GVRD is less than 36 hours, no chlorine residual will remain and regrowth will occur. The improvements resulting from the rechlorination stations are highlighted in Figure 5.

The GVRD transmission system has 17 reservoirs, ranging in size from 2 to 140 ML, which experience periodic episodes of coliform regrowth. Rechlorination did not eliminate this regrowth because of the long retention times in most of the reservoirs. Studies were carried out to determine the minimum level of reservoir turnover that would be required to maintain water quality as measured by HPC and chlorine levels. Once this turnover rate was determined, reservoir-specific strategies were developed to ensure these turnover levels were maintained. The strategies were implemented and have been successful. As long as the water in the reservoirs is turned over at 33–50% of daily rates, HPC and coliform bacterial growth are controlled without further rechlorination. This solved the problem in the large reservoirs in the GVRD system, but a number of the municipalities, such as the city of Coquitlam, also had reservoirs and needed to develop strategies to solve the regrowth in their own systems.

Coquitlam Lake Ozone Disinfection Facility

As part of the program to deal with inadequate primary disinfection of the source water, the GVRD constructed an ozonation facility at the Coquitlam Lake source. This facility was commissioned in May 2000. It was known that the use of ozone can exacerbate regrowth, especially in water that has not been filtered. As part of the decision to proceed with ozone without subsequent filtration, the GVRD consulted several North American water quality and treatment specialists about the potential problem with regrowth. The group stressed that there would be problems unless the GVRD and its members ensured that the following steps were taken:

- Maintain a minimum chlorine residual throughout 90% of the distribution system of 0.2 mg/L.
- Expedite construction of rechlorination stations to achieve the above.
- Expand municipal water main flushing and cleaning programs.
- Regularly exercise distribution system reservoirs to maintain water quality.
- Review and expand water quality monitoring programs.

With these measures it was concluded that any increased potential for regrowth associated with ozone without filtration would be mitigated. Even without ozone disinfection, high coliform bacteria counts would persist unless higher chlorine residuals were maintained in the municipal distribution systems. Characteristics that may affect the level of regrowth

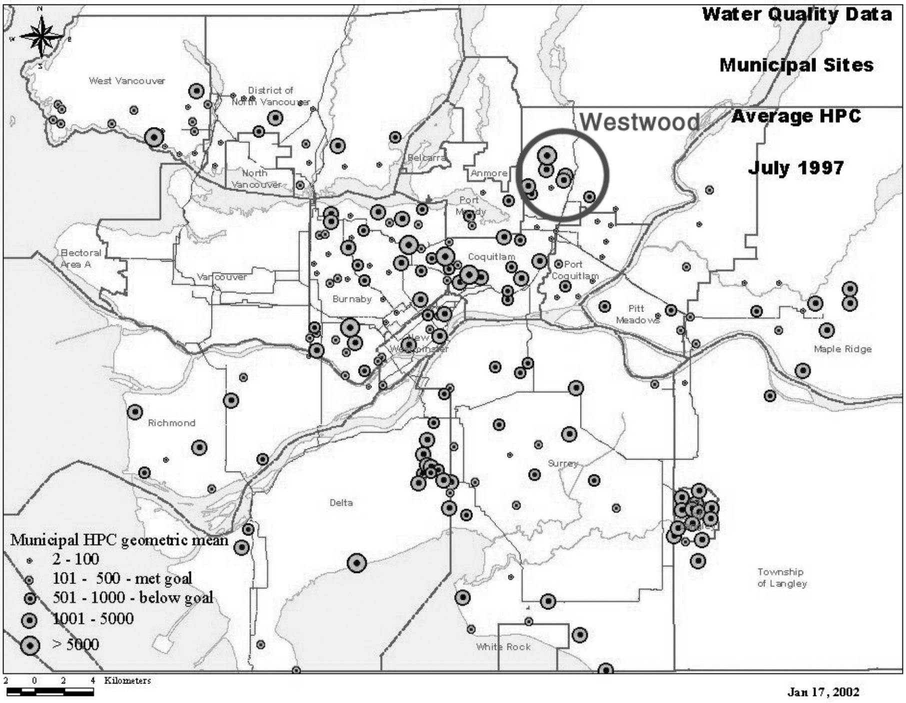


Figure 4 HPC levels prior to GVRD rechlorination

include natural organics in the water supply, water temperature, the presence of unlined cast iron pipes, the corrosive nature of water, the level of assimilable organic carbon or other nutrients, a buildup of debris in the distribution system, and the lack of a disinfectant residual. In the short term it is possible for the GVRD and member municipalities to reduce the effect of at least two of the important factors causing regrowth. The buildup of debris can and has been addressed through flushing and cleaning of the distribution system, and the lack of a disinfectant residual in the distribution system can be addressed through the installation of booster chlorination stations. In the longer term, filtration of the water will remove much of the organic material associated with regrowth.

When the ozonation facility was commissioned, the physical nature of the distribution water changed. The oxygen content increased from near saturation to 150–200% saturation; the UV absorbency at 254 nm dropped to 30–50% of preozonation levels. The initial chlorine demand (10 min) dropped slightly, but the 24-hr demand remained unchanged.

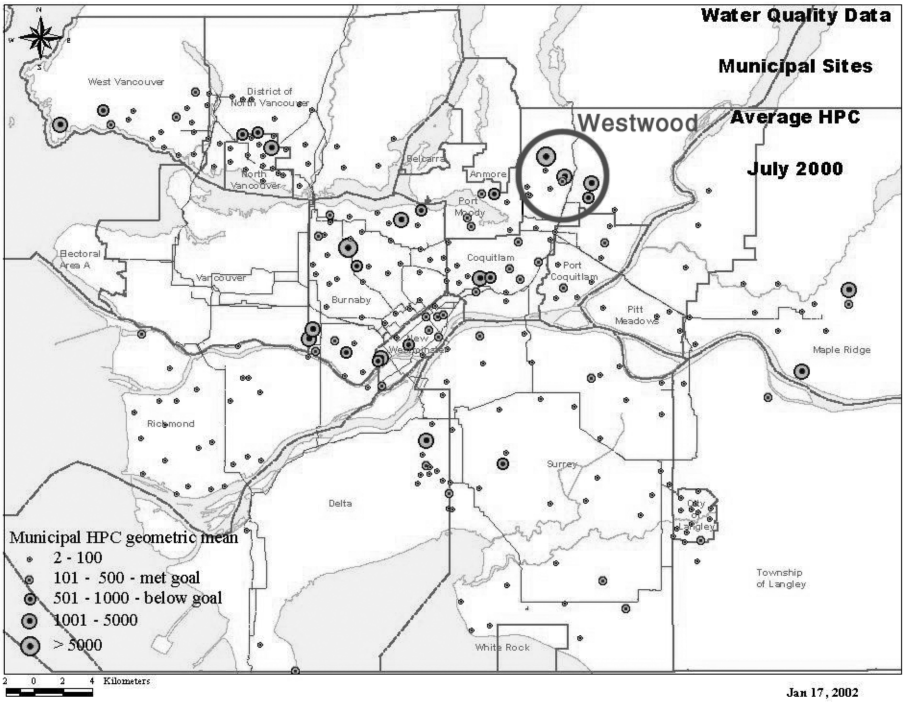


Figure 5 HPC levels subsequent to GVRD rechlorination

As expected, there was an increase in the assimilable organic carbon (AOC) and biodegradable dissolved organic carbon (BDOC) of the treated water (Ferguson et al., 2001).

Pilot studies on Coquitlam Lake water predicted two- to threefold increases in the levels of AOC between source and ozonated water and a threefold increase in the BDOC level (Ferguson et al., 1998). Since commissioning of the ozone facility, the levels in the treated water are similar to the predicted levels, as presented in Table 2.

There was no overall increase in regrowth during 2000 or 2001 in either the GVRD or municipal systems following the startup of the ozonation facility at Coquitlam. The amount of chlorine required to control the presence of coliforms in the municipal distribution system did not change. If the free chlorine residual was higher than 0.1 mg/L, coliforms were controlled. Figure 6 shows the effect on the percentage of samples testing positive for coliforms with varying levels of chlorine residual.

The level of free chlorine present in the water also affects the presence of HPC bacteria. When the free chlorine level is less than

Table 2 BOM in source and treated water

Water	AOC[*] *μg C/L*	BDOC[†] *mg/L*
Untreated water	92	< 0.05
Chlorinated water	280	0.15
Ozonated then chlorinated water	680	0.46

[*] Empirical AOC (P17+NOX)
[†] Bulk BDOC after method of Servais and Billén

0.1 mg/L, a significant portion of samples have greater than 500 cfu/mL. However, the proportion of samples where this occurred did not change following the startup of the ozonation facility, as shown in Figure 7. Free chlorine residuals greater than 0.5 mg/L virtually eliminate the occurrence of HPC counts greater than 500 cfu/mL.

It must be noted that the data presented in Figures 6 and 7 originate from all municipal sampling stations in the region. Under normal operating conditions, different stations receive a mix of water from Capilano Lake, Seymour Lake, Coquitlam Lake, or a combination of these sources.

The amount of chlorine required to control HPC levels in the distribution systems also did not change following the startup of ozonation. There was nothing exceptional about the regrowth in the two years following the startup of the ozonation facility, with the exception of the Westwood Plateau area in the city of Coquitlam. The regrowth at this location will be discussed in the following sections of this chapter.

CITY OF COQUITLAM

Background

The city of Coquitlam is a suburb of Greater Vancouver with a population of 114,000. The city's municipal distribution system delivers water to residential and approximately 650 industrial and commercial customers. Since the city of Coquitlam has no water sources of its own, all distribution system water is purchased from the GVRD.

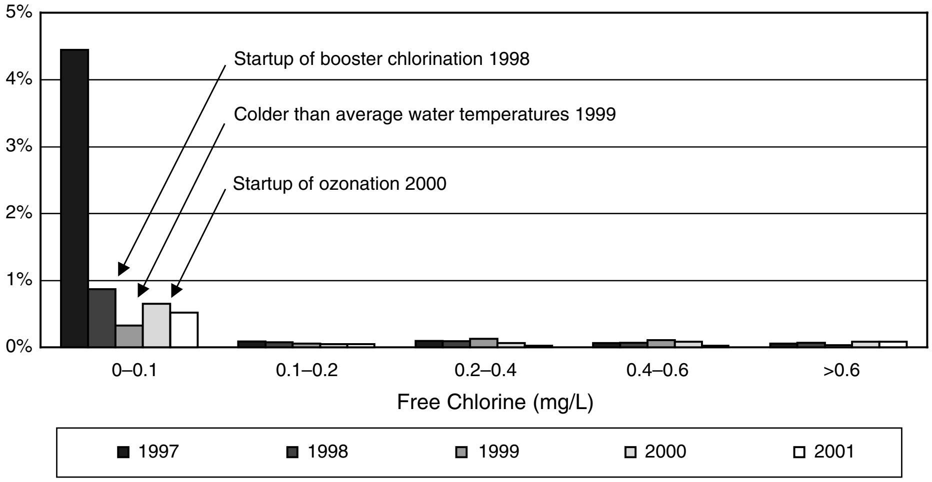


Figure 6 Positive total coliform samples with varying chlorine levels

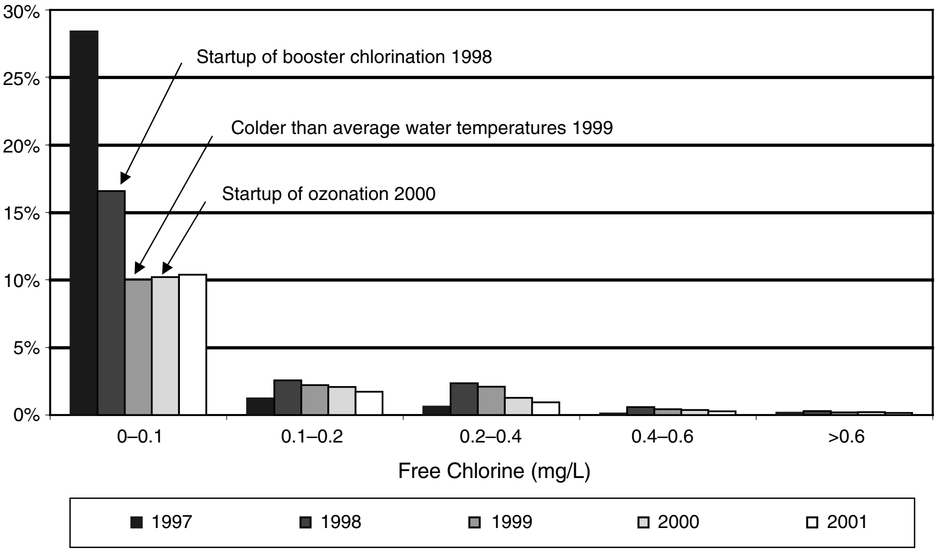


Figure 7 HPC samples > 500 cfu/mL with varying chlorine levels

Prior to 1997, the city of Coquitlam was typical of other GVRD municipal members where, at most sampling stations, the average HPC bacteria counts were in excess of the desired goal of less than 500 cfu/mL.

As noted in the previous section, the installation of rechlorination stations by the GVRD resulted in dramatic improvements, particularly in the outer areas of the GVRD system. However, these stations provided little benefit to the Westwood Plateau area of Coquitlam because they are located farther downstream. Although the city of Coquitlam is divided into several service areas, this chapter focuses on the Westwood Plateau area as described in the next section.

Of the three impounding lakes owned by the GVRD, approximately 50% of the water purchased by the city of Coquitlam originates from Seymour Lake (chlorine-disinfected supply), while the remainder is obtained from Coquitlam Lake (ozone-disinfected supply). As noted earlier, the Westwood Plateau is supplied by water only from Coquitlam Lake, except during rare events when this source is taken off-line.

Westwood Plateau

The Westwood Plateau is a development area supporting approximately 15,000 residents on a mountainous ridge in northwest Coquitlam. The development is relatively new, with the average age of approximately 10 years for water system components (reservoirs and pipes). Currently, the flow to the service area averages 9.7 ML/day and peaks at 17.6 ML/day during the irrigation season (June–September). The distribution system consists of approximately 77 kilometers of water supply piping, with materials throughout the service area consisting of predominantly PVC (53%) and ductile iron with cement mortar–lining (47%).

The service area varies considerably in elevation. The lowest customer served lies at 64 m above sea level and the highest at about 390 m. To maintain operating pressures between 350 and 1,050 kPa, the water supply area is divided into five pressure zones in which each has a reservoir containing sufficient water to balance system demands and for fire-fighting purposes.

Five of the permanent sampling stations are adjacent to reservoirs. The remaining two are at extremities of the lower two zones of the distribution system. Figure 8 provides a schematic view of the arrangement of reservoirs and key pump stations in Westwood Plateau.

The step system layout of the five supply zones and storage tanks results in water that must travel in series through each successively higher supply zone prior to reaching the highest zone's storage tank (Summit Reservoir). The lowest of the pump stations draws water directly from either of the two GVRD supply mains carrying treated water from Coquitlam Lake.

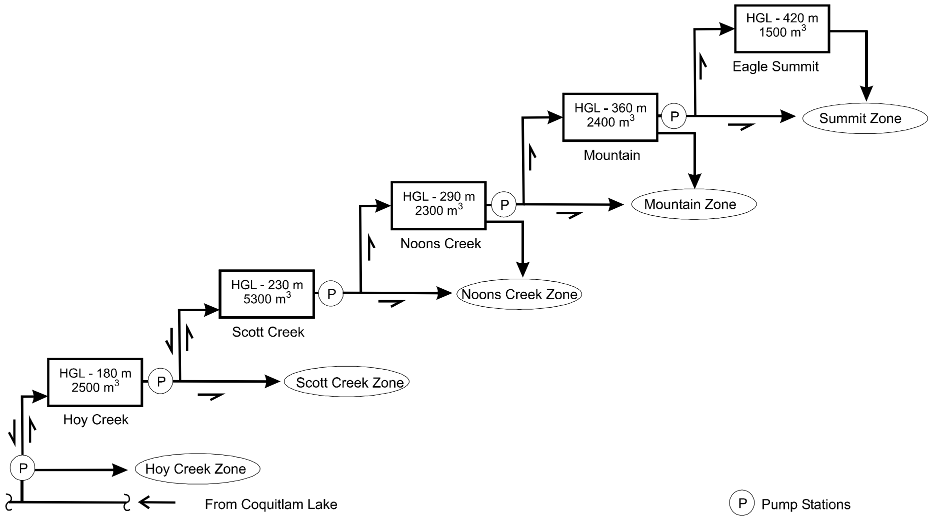


Figure 8 Westwood Plateau system schematic

Historical Perspective

The city of Coquitlam undertakes weekly monitoring of the quality of the water supplied to the public. Samples are taken every week from the permanent stations and tested for total coliform, fecal coliform, HPC bacteria, temperature, turbidity, and free chlorine residual.

The supply point to Westwood Plateau is approximately 6 km downstream from the new water treatment facilities at Coquitlam Lake. The free chlorine residual at this point is typically 0.7 to 0.8 mg/L. Water temperature ranges between 4° and 17°C depending on the season.

Due to the physical layout of the water system and because some service areas had not fully developed, water retention times within the Westwood Plateau distribution and storage system were known to be very long and could exceed two weeks. Figure 9 illustrates the loss of chlorine residual as water travels between the lowest reservoir, Hoy Creek Reservoir (#605), and the highest, Summit Reservoir (#609).

Historically, the Westwood Plateau area experienced regular occurrences of regrowth and sometimes continuous regrowth events with high HPC counts and occasional detection of total coliform. Regrowth had always been a problem in the late summer and early fall when water temperatures were at their highest and summertime water demand was declining. During this period, HPC counts in the tens of thousands were

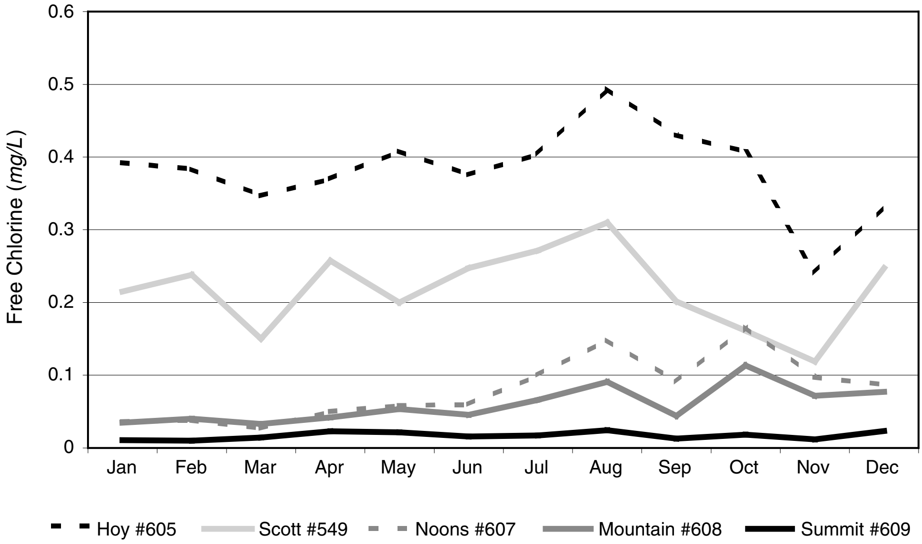


Figure 9 Average free chlorine 1997–2000

not uncommon and were particularly high in areas where water age exceeded several days. Figure 10 shows the average monthly HPC levels for the Westwood Plateau for the period from 1997 to 2000.

The rechlorination stations commissioned by the GVRD to mitigate high HPC counts had no effect in Westwood Plateau since they are all farther downstream in the distribution system. The sites exhibiting high HPC counts were finished water reservoirs.

The city of Coquitlam's normal response to coliform occurrences was to purge the water in question from the system, refill, and resample. This method had been successful in other areas in preventing a recurrence of total coliform. Figure 11 shows the number of positive tests for coliform from 1997 to 2000.

The number of positive total coliform results in September and October 2000 is not typical in the historical sense. The following section focuses on this regrowth event and the issuing of a boil water advisory for the Summit zone in the Westwood Plateau.

Coliform Regrowth in the Fall of 2000

The positive total coliform samples in September 2000 were samples drawn from the extremities of the Westwood Plateau system and included one overgrown sample from the Summit Reservoir (#609). Audit samples

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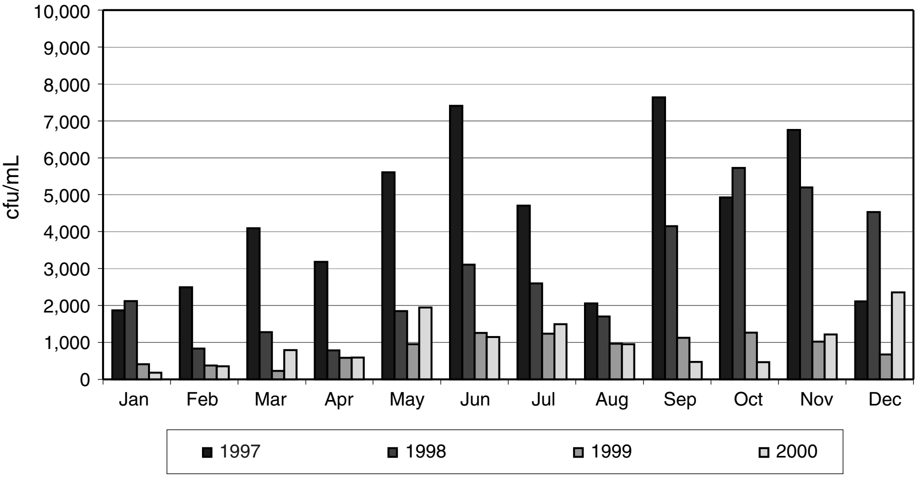

Figure 10 Average HPC counts

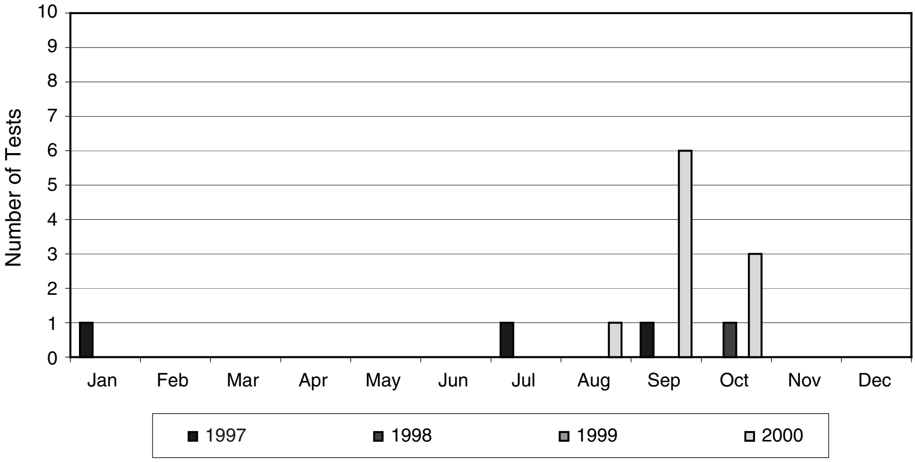

Figure 11 Positive total coliform tests

taken by the local health authority from two restaurants in the area also tested positive for coliform. The city of Coquitlam responded by purging water from the reservoir and flushing the water system in the area. A subsequent overgrown sample, again taken from the Summit Reservoir in late September, caused local health officials to suspect contamination of the water distribution system.

Although no fecal coliforms were detected, the local health officials directed the city of Coquitlam to issue a boil water advisory and take steps to determine the cause and take corrective action. An extensive sampling regime was ordered by the health region and followed through by the city. Although previous coliform incidents had not given rise to a boil water advisory, this response from the health region was not surprising given the tendency throughout Canada to be more cautious after the May 2000 incident in Walkerton, Ontario.

This incident occurred during the typical GVRD regrowth season, which is the late summer and early fall. This is when the water temperatures are the warmest (15°C or higher), the chlorine demand is highest, and the water usage starts to decrease, resulting in longer retention times in the distribution system. Prior to the incident, the chlorine residuals had been very low for several months and HPC counts were routinely exceeding 500 cfu/mL. This, however, was a typical occurrence with the exception that the percentage of HPC counts greater than the 500 cfu/mL goal was higher in 2000 than that from 1999. The only other noted differences compared with 1999 were the new ozone facility, which was commissioned in May 2000, and slightly warmer water temperatures. The recent initiation of ozonation at the Coquitlam Lake source resulted in the production of higher amounts of assimilable organic carbon (AOC) in the water supplied to the city. Figure 12 indicates the monthly breakdown of annual data from Westwood Plateau for the percentage of samples with HPC counts greater than 500 cfu/mL.

As the purveyor of water in the affected area, the city was responsible under provincial law to rectify the conditions that led to the boil water advisory. The city also made itself responsible to its customers. Immediately after publishing the order, the city

- delivered notices to each home and business in the affected area;
- posted signs on the streets leading to the area;
- delivered daily updates advising consumers of the remedial steps taken and the status of the drinking water;
- provided bottled water to all residents in the affected area;
- flushed the water system;
- drained and cleaned reservoirs; and
- manually added sodium hypochlorite at reservoirs to increase free chlorine residuals.

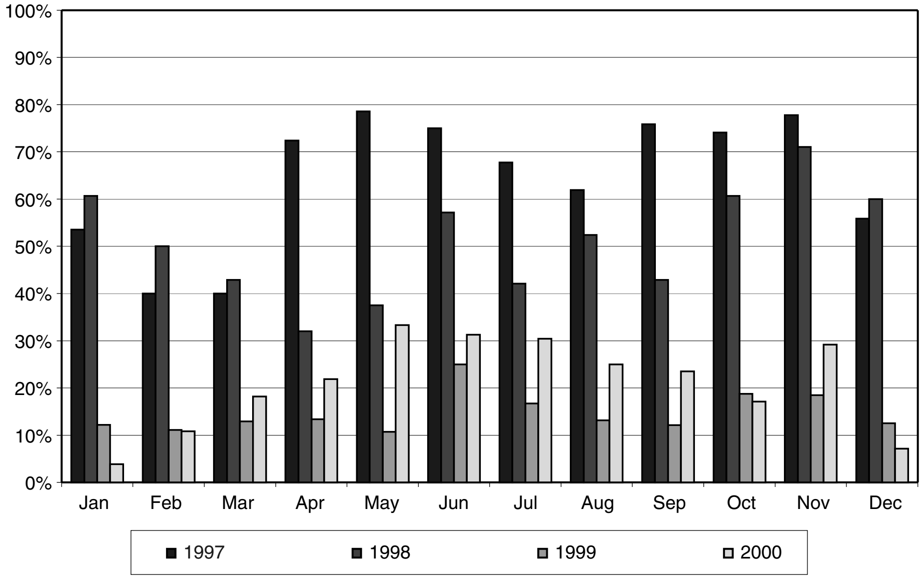

Figure 12 HPC counts greater than 500 cfu/mL

During the boil water advisory, the GVRD assisted the city by

- raising the secondary disinfection level at the Coquitlam Lake source by 0.2 mg/L to assist the city in establishing a chlorine residual in the Westwood Plateau immediately during the boil advisory response;
- providing laboratory services for coliform tests, which totalled an additional 1,000 samples per month; and
- providing expert technical advice to both the city and the local health officials on the nature of the event and the interpretation of test results.

SOLUTIONS TO REGROWTH

Distribution System Operation

Regrowth had been an issue in the Westwood Plateau long before the total coliform occurrences of 2000. There had been no detectable chlorine residual and unacceptably high HPC levels for years. Changes had already been made in the operation of the reservoirs in attempts to improve the quality of the water in the area. Several variables that affect presence and growth of HPC bacteria were considered and evaluated in terms of minimizing regrowth in the Westwood Plateau water distribution system. It was believed that the most significant impact on reducing the regrowth potential would result from lowering the age of water stored in the supply zone reservoirs. The age of the water was defined as the time since leaving the primary treatment facility. A distribution system model can be used to simulate this sequence and to simulate the decay of free chlorine residuals under varying system demands.

Operating characteristics in the Westwood Plateau water system greatly increased the age of water delivered, except during peak summer demands. During the lower-flow periods, demand is typically about 50% of the peak summer demand. Though the balancing storage required in the reservoirs was significantly lower than in the summer, the city kept the reservoirs full throughout the year. The ratio of storage to daily demand was, therefore, unnecessarily high, except during the peak summer conditions.

Uncontrolled timing of reservoir filling also led to higher residence times. As shown in Figure 8, the step system layout of the five storage tanks results in water that must travel in series through each successively higher supply zone prior to reaching the storage tank for the highest zone (Summit Reservoir). Pumping during periods of high demand reduces the inflow of fresh water to the upper zones of the system. It was estimated that between 30% and 70% of water was being consumed enroute to the reservoirs. This loss reduced the daily volume available to refresh the reservoirs.

It was found that by reducing the amount of water stored and by refilling the reservoirs simultaneously during off-peak periods, water age could be significantly reduced. The changes implemented showed a dramatic improvement in the age of water, particularly in the upper pressure zones. However, these changes implemented in late 1998 had a minimal effect on increasing the free chlorine residuals throughout the Westwood Plateau. The age of the water still exceeded the 1.5 days typically taken for the chlorine residual to be consumed by the chlorine demand of GVRD water. However, the reduction in the percentage of HPC above 500 cfu/mL was significant in the Noons (#607) and Mountain (#608) reservoirs, while at the Summit Reservoir (#609), there was only a minor decrease of approximately 5%. The HPC results from stations #605 and #549 were much lower than from sampling stations in the three upper zones even prior to the modifications in 1998. This was because of the relatively low residence time compared to the three upper supply zones. Figure 13 highlights the improvements resulting from the above changes.

The drop in percentage of HPC shown for Summit Reservoir (#609) in 2000 and 2001 was a direct result of the city of Coquitlam's response to the regrowth event in the fall of 2000. The response during this regrowth event included flushing of water mains, cleaning of reservoirs, and manual rechlorination of the three upper supply zone reservoirs. The manual rechlorination was continued until the permanent automated rechlorination facilities were constructed in June 2001.

Distribution System Maintenance

The city of Coquitlam has always flushed dead ends annually but has not undertaken unidirectional flushing of the water mains in the Westwood Plateau because it was concentrating on the older, more problematic areas in the city and, historically, coliform readings had not been a problem in the Westwood Plateau area. Since the event of September 2000, the city has been working toward unidirectional flushing of the entire Westwood Plateau on a three-year cycle. All of the reservoirs in this area have been cleaned of accumulated sediment and biofilm and disinfected since the event. Based on experience in older areas of the city,

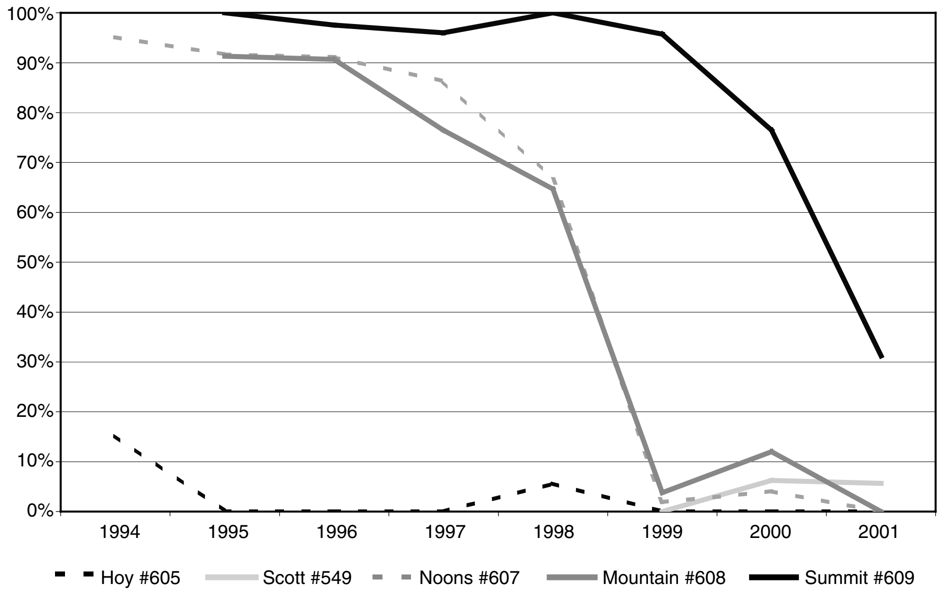

Figure 13 Samples with HPC > 500 cfu/mL

it is anticipated that these measures will reduce chlorine demand, and thus improve free chlorine residuals.

Maintaining a Chlorine Residual

The operational changes made up to 1999 had slightly improved the free chlorine residuals in the lower supply zones of the Westwood Plateau, but they had not significantly improved the free chlorine levels in the three upper supply zones (Noons, Mountain, and Summit). The low system demand in relation to the reservoir storage volume continued to result in excessive water age.

As part of the work program initiated in response to the regrowth event in late 2000, a hydraulic model, using extended time simulation, was used to evaluate the maintenance of a chlorine residual in the Westwood Plateau area. Based on this modelling, it was determined that one rechlorination station would be required at the Scott Creek pump station for water supplied to the Noons Creek Reservoir (#607), which in turn supplies the Mountain (#608) and Summit (#609) reservoirs. It was shown that this rechlorination station would be sufficient to provide a free chlorine residual greater than 0.2 mg/L throughout the upper zones during the peak summer demand period. However, the model also showed that at the end of the peak summer demand in mid- to late September, the reduced water consumption resulted in the loss of the chlorine residual in the Summit Reservoir (#609). A second rechlorination facility at the Eagle Mountain pump station increases free chlorine residuals in the Summit Reservoir during the off-peak demand period, particularly during the remainder of the high regrowth season.

Figure 14 shows the average annual free chlorine residual in the Westwood Plateau zone reservoirs for the period from 1993 to 2001. The continuous improvement shown for the Hoy Creek Reservoir (#605) is the result of ongoing expansion of the new development. The operational control changes in late 1998 show corresponding chlorine residual improvements in 1999 results. A higher chlorine residual for 2000 is the result of manual addition of sodium hypochlorite at the four upper supply zone reservoirs. The automated rechlorination stations were both installed in 2001 and resulted in substantial improvement in the free chlorine residuals.

One of the concerns with rechlorinating is the creation of disinfection by-products, particularly trihalomethanes (THMs) and haloacetic acids (HAAs). The Guidelines for Canadian Drinking Water Quality (GCDWQ) recommend total THMs in drinking water not exceeding

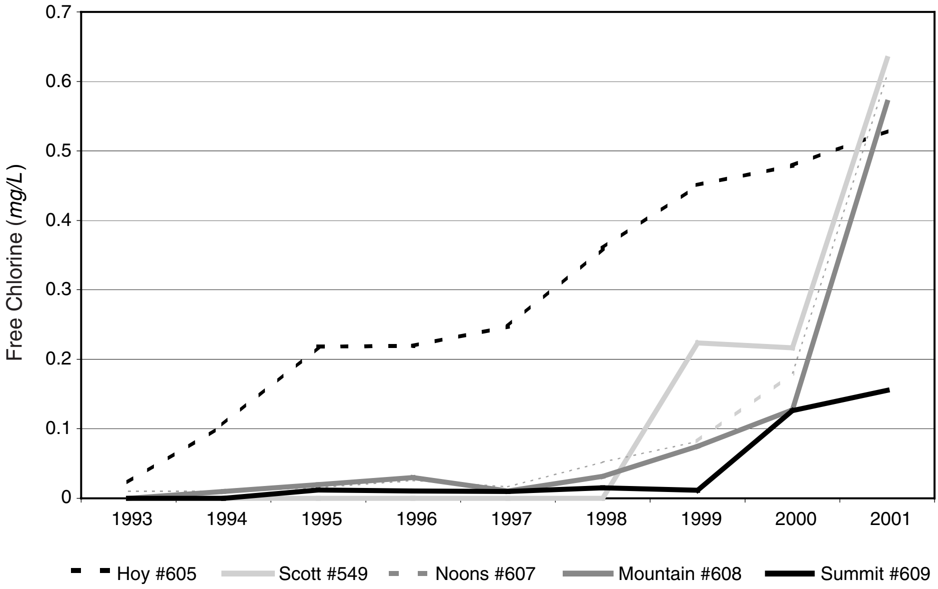

Figure 14 Average free chlorine from 1993 to 2001

0.1 mg/L, based on an annual average of quarterly samples. Currently, there are no Health Canada guidelines for total HAAs.

The city of Coquitlam has monitored the levels of THMs and HAAs in the Summit Reservoir since early 2001. Tables 3 and 4 summarize the results of the testing. Table 3 indicates that THMs in the Summit Reservoir are lower than the maximum levels recommended in the GCDWQ. The HAA levels are higher in the upper zone of Westwood Plateau since rechlorination has been initiated.

Optimization of the target free chlorine levels at the two rechlorination stations may reduce both the THM and HAA levels in the Summit Reservoir (#609).

CONCLUSIONS

The city of Coquitlam purchases all of its finished water from a wholesale distributor, the Greater Vancouver Regional District. In the fall of 2000, the city of Coquitlam had a water quality issue in Westwood Plateau. High levels of total coliform bacteria were found in samples collected from a number of locations in this localized area, an area with a history

Table 3 THM monitoring results

Date Collected	Bromodichloromethane	Bromoform	Chlorodibromomethane	Chloroform	Total THMs
14-Mar-01	1.3	<1	<0.5	36	37
20-Jun-01	1	<1	<0.5	31	32
15-Aug-01	0.7	<1	<0.5	25	26
21-Nov-01	1	<1	<0.5	19	20
07-Mar-02	1.1	<1	<0.5	28	29

Table 4 HAA monitoring results

Date Collected	Dibromoacetic Acid	Dichloroacetic Acid	Monobromoacetic Acid	Monochloroacetic Acid	Trichloroacetic Acid	Total HAAs
14-Mar-01	0.6	10	<1	5	33	49
20-Jun-01	<0.5	4	<1	7	31	42
15-Aug-01	<0.5	28	<1	14	36	78
21-Nov-01	<0.5	5	<1	7	47	59
07-Mar-02	<0.5	12	<1	<5	40	52

of low chlorine residuals and high heterotrophic plate counts. The local health region advised the city to issue a “boil water advisory” to the residents and businesses in this area. Staff from the city of Coquitlam and the GVRD worked together as follows:

- GVRD staff assisted the city with knowledge and expertise in the area of water quality and bacteriology;

- the city arranged to collect additional water samples that the GVRD laboratory analyzed (at no direct cost for the approximately 1,000 extra samples in a month);
- the chlorine residual leaving the source was raised by 0.2 mg/L; and
- meetings were held among the GVRD, city of Coquitlam, consultants, and health officials.

The GVRD and the city of Coquitlam have a partnership that works to deliver a quality product to consumers. Given the timing of this problem and the startup of the ozonation facility, it would have been easy for the city to simply blame the GVRD and source water for the problem. An easy response by the GVRD would have been to refer to previous information, which showed that the Westwood Plateau area needed a free chlorine residual and that the reservoirs had a water quality problem. A quick resolution to the situation was achieved because both sides recognized that a cooperative effort was required.

The relationship between the water quality staff at the city and the GVRD had been built through GVRD-sponsored events like workshops on water main flushing, discussions between lab and municipal staff on laboratory results, and discussions among groups when changes in source water quality affected the quality at the tap. The relationship was good before the boil advisory, with a high level of trust between the two groups. When a crisis did occur, the two groups worked together to help find a solution. Of significance, the total cost to the city of Coquitlam for the response to the incident was approximately $340,000, while the cost of installation of two rechlorination stations was $240,000.

The most likely source of the coliform bacteria was determined to be regrowth. Since one of the major contributing factors was the lack of a significant chlorine residual, the solution was to increase the chlorine residual in the water supplying this area. The cause of regrowth during September 2000 could not be attributed to one particular factor. A combination of conditions provided an environment that was conducive to bacterial regrowth. The conditions that contributed to the presence of excessive HPC bacteria and the detection of total coliforms in the Westwood Plateau distribution system were as follows:

- excessive water age;
- free chlorine residuals lower than 0.2 mg/L;
- seasonal variations in water temperature;

- presence of AOC in excess of 100 μg/L;
- infrequent cleaning of reservoirs; and
- inadequate flushing of water mains.

The city of Coquitlam has found that regrowth in the distribution system can be controlled if an adequate level of free chlorine is maintained in the system. Review of water quality data from the Westwood Plateau and throughout the region has indicated that HPC bacteria and coliforms are minimized with free chlorine residuals greater than 0.2 mg/L.

The following strategies were successfully implemented by the city of Coquitlam to increase free chlorine residuals and control regrowth:

- reduction of water age through optimization of operating conditions;
- unidirectional flushing of distribution piping and cleaning of reservoirs on a three-year cycle; and
- rechlorination at two strategic locations in the distribution system.

These strategies have controlled the regrowth of HPC bacteria and, to date, have eliminated the occurrence of total coliform bacteria in the Westwood Plateau. Monitoring of the HPC levels in the reservoirs provided a useful tool in determining the effectiveness of approaches that the city had taken to reduce the occurrence of regrowth.

Rechlorination of water in Westwood Plateau has not significantly increased the level of THMs; however, initial results show that HAA levels in the Summit Reservoir have increased above values requiring removal by treatment in other jurisdictions. In the short term, the city of Coquitlam gained control over historically high HPC counts. The future addition of filtration at the Coquitlam Lake ozone facility will help in the long term to improve the biological characteristics of the drinking water. The reduction of chlorine demand resulting from the implementation of filtration would improve free chlorine levels throughout the region and decrease the requirement for rechlorination at certain locations. The reduced requirement for rechlorination will result in a reduction of disinfection by-product formation.

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CHAPTER · 7

Distributed Water Quality After Three Years of Nanofiltration

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INTRODUCTION

Producing biologically stable water is difficult because finished water still contains dissolved organic carbon (DOC) and bacteria, both of which allow biofilm accumulation in distribution systems. Moreover, the reaction of chlorine with DOC also allows the formation of trihalomethanes (THMs) at the end of the treatment and in the network. Due to its small pore size, nanofiltration allows a much higher removal of DOC than conventional processes. Hence, the level of DBP precursors and biodegradable DOC (BDOC) are reduced. On this basis, the Suburbs of Paris Utility decided to implement a large surface water nanofiltration unit to improve the quality of its distributed water. At the Méry-sur-Oise Water Treatment Plant (90 mgd capacity), nanofiltration was put in service in fall 1999 as a refining step after flocculation, ozonation, and dual media filtration. The distribution system of the northern suburbs of Paris, now fed with a blend of nanofiltrated water (70%) and conventionally produced water (30%), is a large network with many storage tanks and booster chlorination units, delivering the water to over one million people.

An ongoing research project was set up to document the impact of such a change on the water quality over a period of five years (two years before and three years after nanofiltration). Water quality parameters were followed on 15 sampling stations (public distribution system and service lines), including fecal indicators, total bacteria direct counts (TDC), viable bacteria counts (VC), DOC, and BDOC. Physico-chemical parameters include THMs, chlorine residual, turbidity, and temperature. This chapter presents and discusses data obtained during this survey.

CONTEXT AND OBJECTIVES

Reducing DOC in drinking water is essential to minimize THM formation (Sohn et al., 1997, Westerhoff et al., 2000) and biological instability of the water during distribution (LeChevallier et al., 1991; Van der Kooij, 1992; Servais et al., 1992). In the past 15 years, biological filtration has become a classic way to reduce DOC during water treatment (Bablon et al., 1988; Servais et al., 1991, 1992). Nowadays, nanofiltration (NF) appears to be the most efficient way of minimizing the DOC concentration at the plant outlet. The water produced by NF has very low DOC concentrations and a BDOC content lower than the detection limit, i.e., 0.1 mg C/L. Previous information on the impact of NF on water quality in distribution systems was obtained from pilot studies (Sibille et al., 1997) and a full-scale experiment conducted since 1993 in Auvers-sur-Oise (a 6,500-habitant community). Results indicated that the use of NF allows reducing chlorination of treated water down to 0.3 mg Cl_2/L and at least maintaining or increasing the microbiological quality of the distributed water (Randon et al., 1995; Laurent et al., 1999).

On the basis of these results, the Syndicat des Eaux d'Ile de France (SEDIF), which is responsible for water distribution in the Parisian suburbs, decided to build a large-scale NF unit in Méry-sur-Oise in order to improve its total production capacity and the quality of the distributed water. This NF unit, with a production capacity of 140,000 m^3/d (37 mgd), was started up in September 1999.

In order to document the impact of NF on distributed water quality, a large survey beginning more than one and a half years before the modification of the treatment was performed in the distribution system fed by the plant. Physico-chemical and bacteriological water quality was followed in 15 stations of the distribution system. This study was initially designed to cover two years of operations with NF, but it was decided to pursue the study, which is still under way.

Based on results obtained before NF (1998–1999) and since NF was in service (2000–2002), the objectives of the present study were

- to directly appreciate the impact of NF on (i) THMs and (ii) bacteria numbers; and
- to estimate the relative influences of variables influencing water quality in distribution systems, namely temperature, oxidant residual (i.e., free chlorine), BDOC, and TDC at the inlet to the system.

MATERIALS AND METHODS

The Méry-sur-Oise Water Treatment Plant and NF unit

The Méry-sur-Oise water treatment plant (MWTP) treats Oise River water and has two trains:

- A conventional train (coagulation, flocculation, settling, biological sand filtration, ozonation, biological granular activated carbon filtration, and chlorination) with a nominal capacity of 140,000 m^3/d and average output of 60,000 m^3/d.
- An NF treatment train: clarification (Actiflo), intermediate ozonation, biological dual media filtration (anthracite and sand), cartridge filtration, NF, medium-pressure UV disinfection (400 J/m^2). The nominal capacity of this train is 140,000 m^3/d.

Up to September 1999, the distribution system was supplied only by the conventional train. The new train was started up in September 1999 and reached its full capacity in April 2000. From this point onward, the water feeding the distribution system has been a mixture comprising 30% coming from the conventional train and 70% coming from the NF train. The water currently produced by both treatment trains is blended, adjusted for pH, and dechlorinated.

The NF unit is composed of eight independent files. Each file is composed of three filtration stages, all of them designed to filter half of the inlet flow, with a total yield of 85% in water. Each file has a production of 17,500 m^3/d so that the total production capacity of the unit is 140,000 m^3/d. Each file is composed of 190 tubes with six modules of NF membranes[*] (molecular weight cutoff: 200 D, theoretical DOC removal: 90%). Total surface membranes are around 340,000 m^2.

Sampling and Analysis

The distribution system supplied with MWTP water covers 31 cities in the northern suburbs of Paris, and supplies water to over one million people. There are nine one-line storage tanks (all covered) and 12 booster chlorination units. The distribution system is divided into three main pressure zones. The average residence time is 40 hours, storage tanks not

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[*] Filmtec NF200-B

included. The majority of DS pipes are cast iron, although mains are concrete pipes, and PEHD is now used for small-diameter pipes.

Six sampling campaigns were done each year, with the exception of 2002 (only three sets of results are available).

Fifteen sampling stations were chosen in order to represent various situations in terms of residence time and hydraulic characteristics, including the plant outlet, and the inlet and outlet of the largest one-line storage tank. For each sampling, the water was collected after a flush at low flow over a period of 5 minutes in order to discharge stagnant water. Samples were sent to the laboratory immediately after collection. The analytical methods are listed in Table 1.

RESULTS

MWTP Finished Water Quality

TOC in MWTP finished water is presented in Figure 1. It can be observed that TOC levels fluctuated before September 1999, reflecting DOC changes in the Oise River. After September 1999, the NF trains were progressively started up and the unit reached full capacity in April 2000. The DOC concentration levels decreased to an average of 0.7 in 2000–2002. This value is explained by the blend of 70% of NF water, with an average DOC of 0.2 mg C/L, and 30% of conventional water, with an average DOC concentration of 2.0 mg C/L. An increase in winter is still observable since NF is in service, without reaching values obtained before introduction of the NF treatment.

The BDOC content of the finished water is presented in Figure 2, showing levels around 0.45 mg C/L before NF, although the graph shows a high variability in data. Values are near or under its detection limit, i.e., 0.2 mg C/L, since the end of 1999.

Figure 3 shows the free chlorine residual at the outlet of the plant. Before the introduction of the NF treatment, free chlorine residuals were set between 0.3 mg Cl_2/L in winter and 0.45 mg Cl_2/L during the warm water season. This concentration was progressively decreased as the proportion of nanofiltered water in the finished water increased, down to 0.2 mg Cl_2/L. Since October 2001, 0.3 mg Cl_2/L is mandatory in France at the inlet to any distribution system, as a counterterrorist step.

Finally, it is worthwhile to present the impact of nanofiltration on water hardness, this parameter also being observed by consumers (Figure 4).

Table 1 Analytical methods

Parameter		Method
Total Direct Counts	TDC	Epifluorescence microscopy after DAPI staining (4,6-diamino-2-phenylindole)
Viable Counts	VC	Epifluorescence microscopy after CTC staining (chlorure de 5-Cyano-2-3Dytolyl-Tetrazolium)
Coliforms and Streptococcus		NF T 90-414 and NF T 90-416 (France)
Total and Dissolved Organic Carbon	TOC/DOC	Dorhmann DC-180 after filtration on a 0.45-μm filter
Biodegradable Dissolved Organic Carbon	BDOC	Joret and Levi method (1986)
Trihalomethanes	THMs	NFT 90-125 (France)
Alkalinity		NF 90-036 (France)
Calcium and Magnesium		ISO 11885
Conductivity		NF EN 27888 (France)
pH	pH	320 SET
Total and Free Chlorine Residual Concentrations	TCl_2/FCl_2	HACH POCKET
Turbidity		HACH 2100 P

The change in quality of the finished water had a clear effect on the chemical quality in the distribution system. In fact, conductivity and alkalinity have decreased since the NF unit has been in operation, with averages near 260 mg $CaCO_3$/L before nanofiltration and 170 after. The aluminum concentration level is lower than the detection limit of the method due to the removal of ions by the membranes (results not shown).

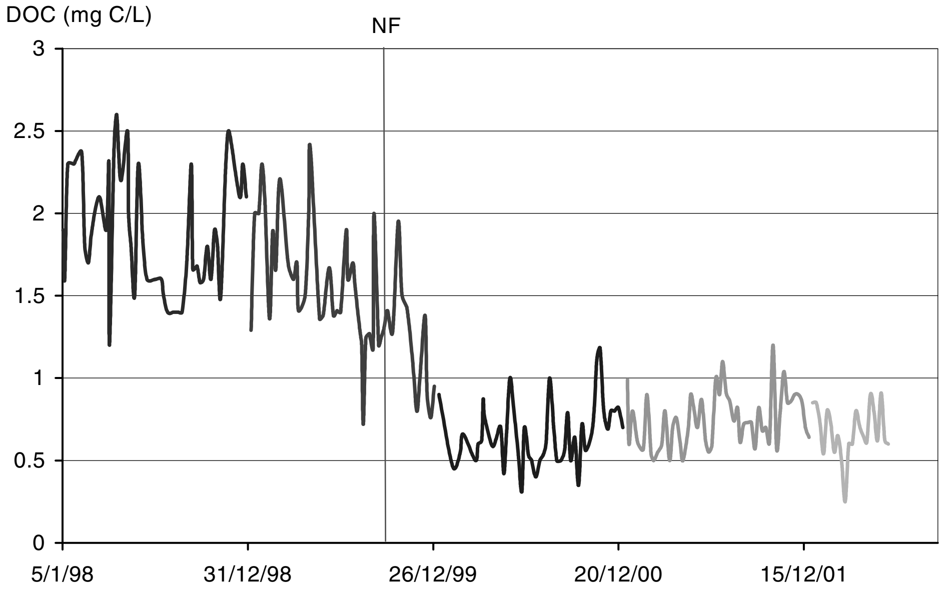

Figure 1 TOC in MWTP finished water (on-line analysis)

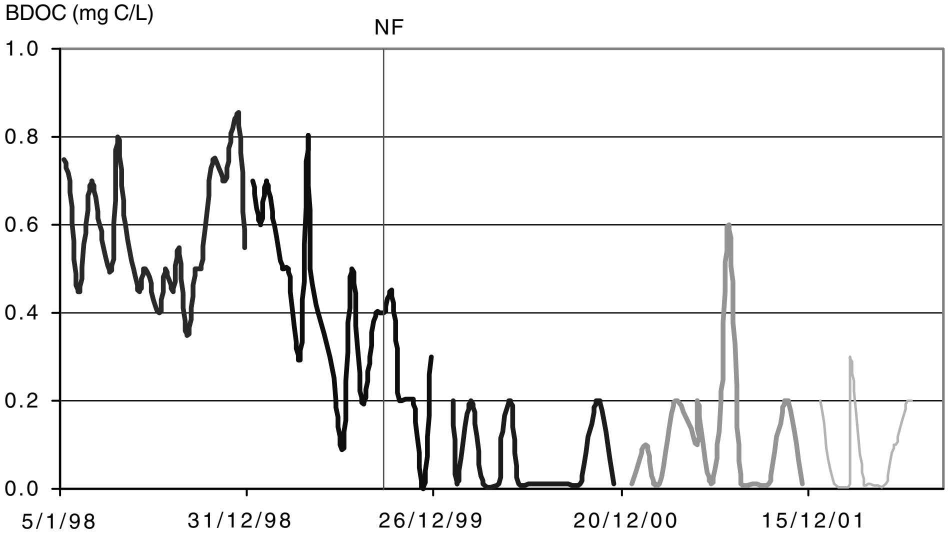

Figure 2 BDOC in MWTP finished water

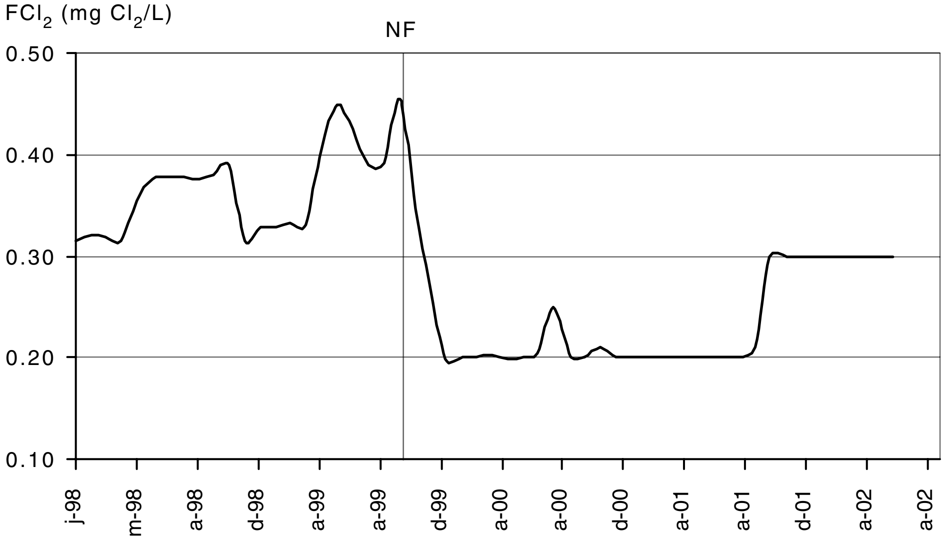

Figure 3 FCl_2 in MWTP finished water

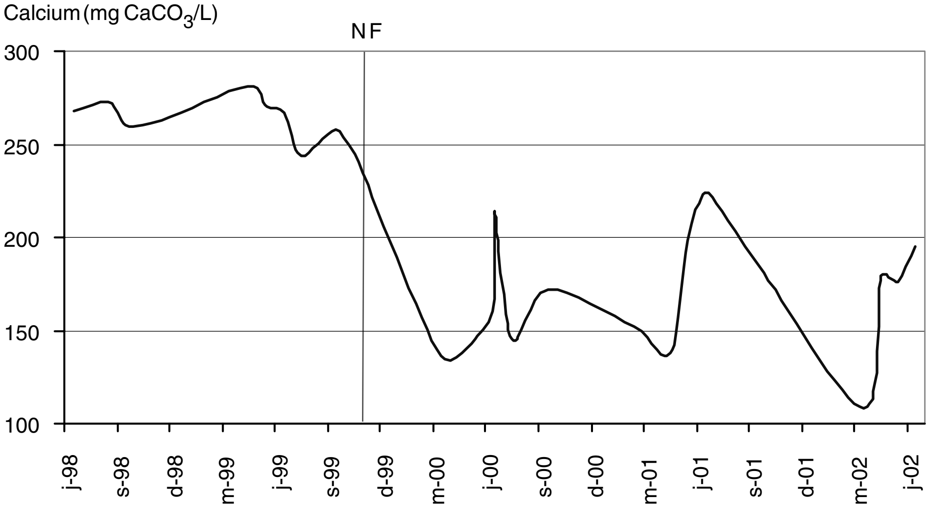

Figure 4 Calcium in MWTP finished water

Distributed Water Quality

Distributed water quality results are presented in box plots, where the boxes represent the range between the 25th and 75th percentiles, and the whiskers the range between the 5th and 95th percentiles.

With regard to free chlorine residuals in the network, a clear modification can be observed in Figure 5. The mean value has decreased from 0.19 and 0.25 mg Cl_2/L to 0.15 and 0.16 mg Cl_2/L since NF has been in service. It can be observed that values were extremely heterogeneous in 1998 and 1999, with excessive chlorine residuals observed at some sampling stations located downstream of booster stations. After the introduction of the NF treatment, there were fewer values with no chlorine residual and fewer values with excessive chlorine residual.

In 2002, the free chlorine residuals were higher as a consequence of the higher set point at the plant outlet.

The decrease in chlorine residuals and DOC in the finished water allows an appreciable reduction of THMs (sum of the four individual species), as shown in Figure 6. While averages were close to 20 µg/L before the introduction of NF treatment (thus already in compliance with the current European standard of 100 µg/L), THM concentrations are now between 8 and 10 µg/L in the distribution system, showing a reduction of 50%. One may observe that the increase in FCl_2 in 2002 did not lead to much higher values. Since NF treatment has been in service, no THM result exceeds 20 µg/L. These results confirm that NF efficiently removes THM precursors, which is important given the potential introduction of more stringent standards.

The other focus of the study is the microbiological quality of distributed water. Sampling did not reveal the presence of any fecal indicator. TDC and VC are presented in Figures 7 and 8.

The TDC reveals that the suspended bacterial biomass in the system decreased gradually from 50,000 cells/mL before the startup of the NF plant to 30,000 the first year after, and to 20,000 the second year. Since the beginning of 2002, despite (or as a result of?) the higher chlorine residuals, the number seems to be closer to 30,000, with a larger scattering of data. Average values are nevertheless significantly lower than before NF.

VC allows one to appreciate the viability of bacteria cells. Average VC was 4,000 cells/mL, with very high values up to 16,000 cells/mL before the NF treatment. It is worth noting that a sharp decrease was observable in 1999, which can only be explained by the higher chlorine residuals in 1999. Data are even lower since NF treatment has been in operation; the average has decreased to 400 cells/mL, with median values

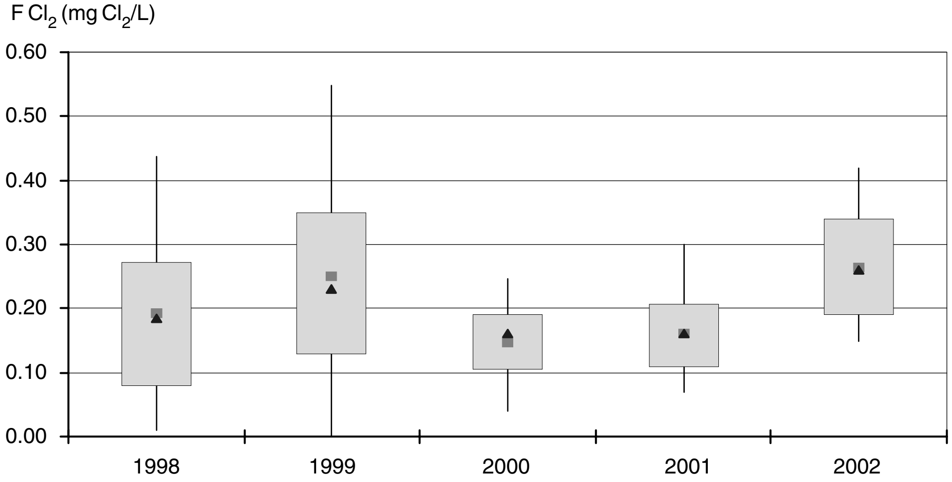

Figure 5 FCl_2 in the distribution system (squares: average; triangles: median)

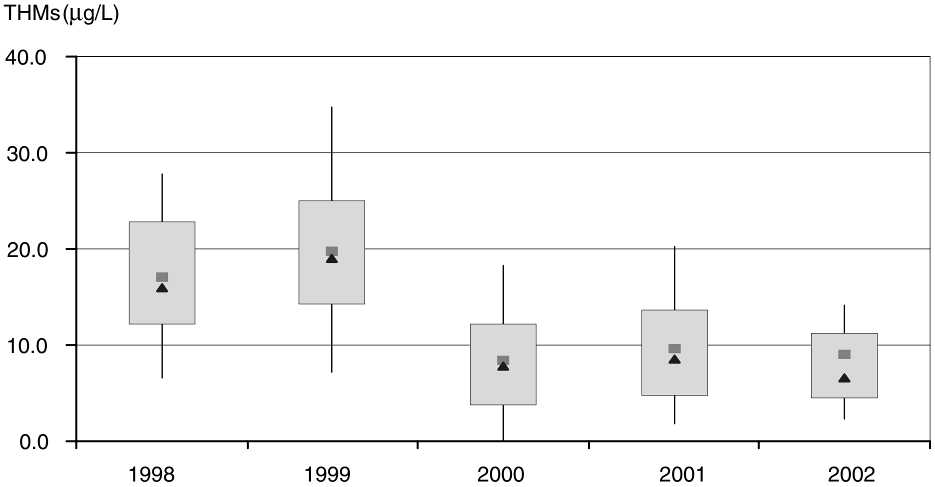

Figure 6 THMs in the distribution system (squares: average; triangles: median)

below 50 cells/mL. These data roughly indicate a 1-log decrease in the number of viable bacteria in the distribution system, with a further improvement in 2002 due to the higher chlorine residuals.

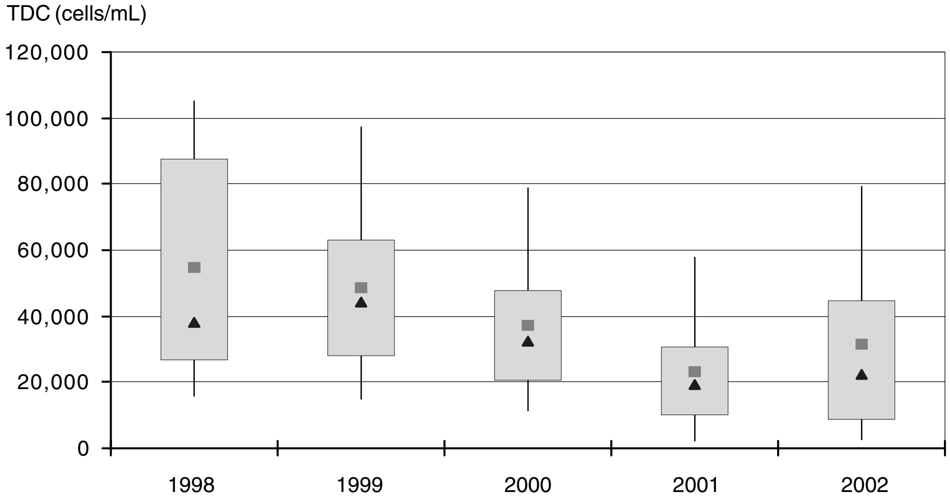

Figure 7 TDC in the distribution system (squares: average; triangles: median)

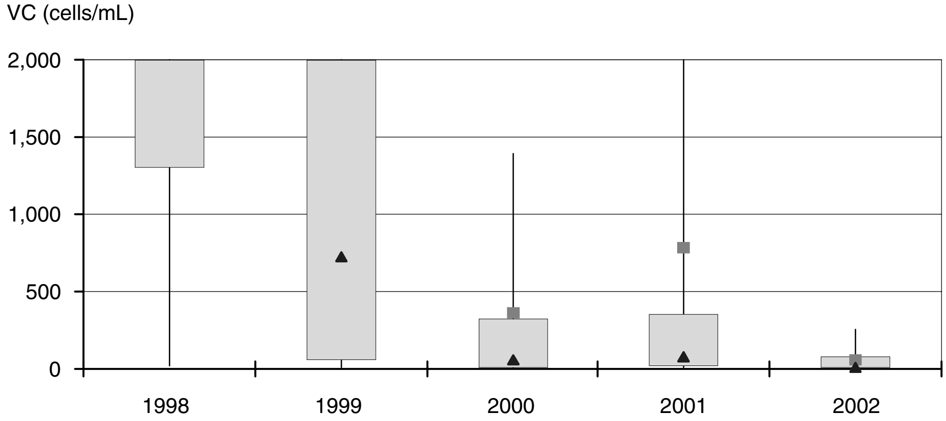

Figure 8 VC in distribution system (squares: average; triangles: median)

DISCUSSION

NF treatment can improve the distributed water quality through a much lower DOC or BDOC concentration in the system. In fact, BDOC values are now under 0.2 mg C/L, to be compared with the threshold for biostability earlier proposed by Servais et al. in 1995 (0.15 mg C/L using the Servais method). This situation leads to a lower biomass, both in the liquid phase and at the pipe interface (biofilm). Results presented here support earlier findings proving that the bacteria numbers in the water phase are now limited to 3×10^7 cells/L; despite potential differences in laboratory counts, this order of magnitude is consistent with pilot studies conducted by Sibille et al. (1997), providing a count of 5×10^7 cells/L. Some biofilm bacteria counts conducted in the supply system of Méry-sur-Oise gave results of 2 to 6×10^6 cells/cm^2, also consistent with the same study. This situation contrasts with the earlier data obtained on the same system, where cell counts in the liquid phase ranged from 3×10^7 cells/L to 9×10^7 cells/L, thus showing a reduction by a factor of 2 at least. Another fact, which cannot be illustrated here, is the relatively smaller size of the remaining bacteria cells, after nanofiltration treatment.

Bacteria counts using CTC staining also show an appreciable decrease in cell counts, linked with either the lower DOC content or the more stable free chlorine residuals; with a median of 700 cells/mL in 1999 and 30 cells/mL in 2002, the proportion of viable bacteria in the liquid phase dropped from 1% to 0.1% after three years of nanofiltration.

A statistical analysis of the database was carried out to identify the stronger trends through the water quality changes related here. Hence, the authors tried to explain TDC averages for each campaign as a function of average free or active chlorine residuals, DOC or BDOC, temperature, and finally TDC at the inlet to the system. The only significant variables are DOC and TDC at the inlet to the system. The best fit to the average TDC in the distribution system has the following equation:

$$\log(\mathrm{TDC}) = 0.37 \log \mathrm{TDC}_{\mathrm{Plant}} + 0.43 \log \mathrm{DOC} + 2.81 \ (R^2 = 0.6)$$

This equation is illustrated in Figure 9. It can be noted that neither the temperature nor the oxidant residual appears significant in this database, although the relatively narrow and low range for chlorine residuals may explain this.

The same work was done for average THMs for each campaign, to be explained as a function of average temperature, DOC, pH, and free chlorine residual. THMs are best explained by the following equation:

$$\log(\mathrm{THM}) = 0.57 \log \mathrm{temperature} + 0.9 \log \mathrm{DOC} + 0.41 \ (R^2 = 0.79)$$

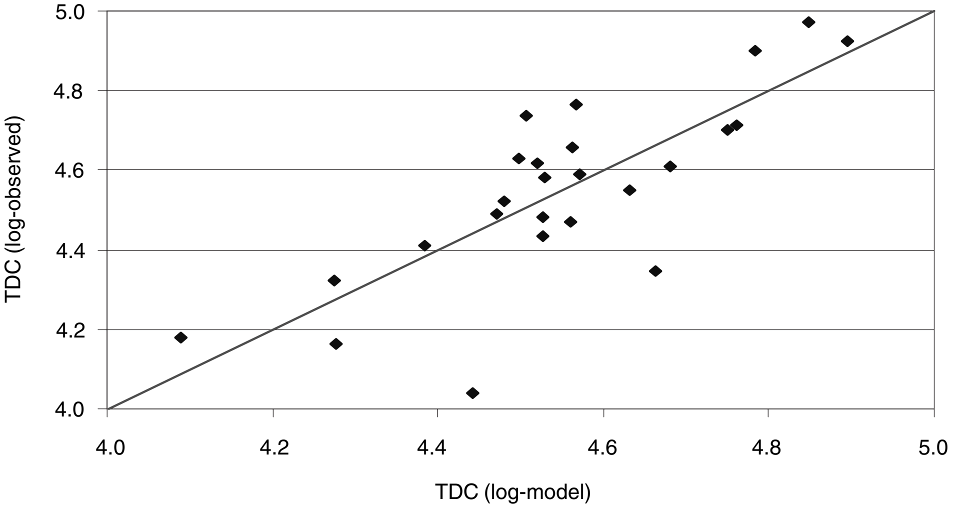

Figure 9 TDC model

This equation is illustrated in Figure 10. In this case also, free chlorine residuals fail to explain part of the variability of data.

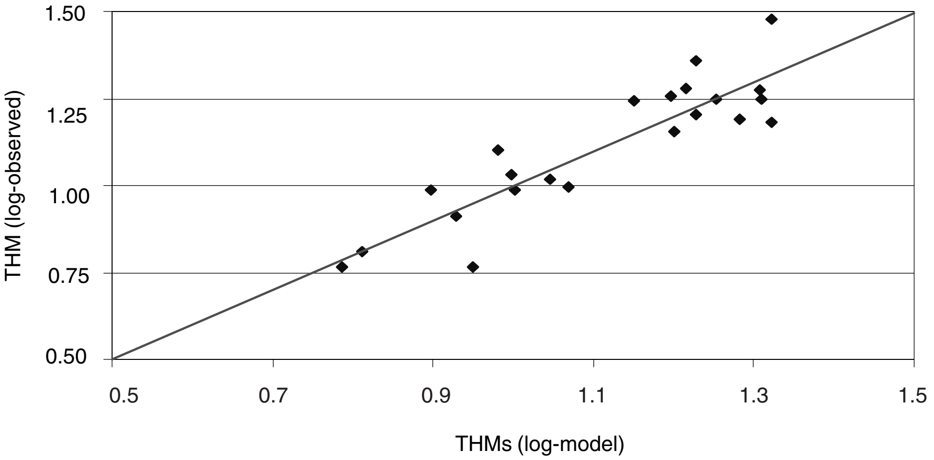

Figure 10 THM model

CONCLUSIONS

The data presented in this paper build on previous experience, demonstrating the capability of NF treatment to produce low-DOC water from surface water. Results presented here show that the chlorine demand of the distributed water was decreased accordingly, allowing the operators to lower the free chlorine residual to 0.2 mg Cl_2/L at the outlet of the water treatment plant. Despite this low residual at the end of the treatment, free chlorine residuals remain stable throughout the system. This situation is beneficial for water taste, since chlorine consumers can perceive residuals above 0.3 mg Cl_2/L.

One main advantage of the decrease in chlorination level and DOC content is the lower formation of THMs, which decreased by more than 50%.

The biodegradable fraction of DOC (BDOC) was reduced after NF treatment to values lower than the thresholds for biostability. Thus, regardless of the level of chlorination, the average microbiological quality of the water improved after the introduction of the NF treatment. In addition, the NF treatment allowed an increase in the physico-chemical water quality: reduction of alkalinity, turbidity, aluminum, and atrazine concentration.

The statistical analysis of data reveals that the main explanatory variable is DOC, for both total direct counts and THMs.

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CHAPTER · 8

Case Study: Ozonation and Distribution System Biostability

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BACKGROUND

Biodegradable organic matter that is not removed during water treatment can lead to the proliferation of bacteria within the distribution system. This, in turn, can decrease water quality, accelerate corrosion rates of pipes, and increase the incidence of bacteriological diseases. The main objective of this project was to collect and analyze full-scale system data on short-term and long-term responses to the implementation of ozonation with respect to its impact on bacterial regrowth potential as quantified by assimilable organic carbon (AOC) and biodegradable dissolved organic carbon (BDOC). A major observation made during the sampling period was that ozonation caused a significant increase in the AOC concentration of the distribution system (more than 200% in the short term and more than 100% in the long term). Use of ozone also caused a significant increase in the bacterial counts within the distribution system for both the long and short terms. In contrast, ozonation affected BDOC and dissolved organic carbon (DOC) concentrations only slightly compared with its impact on AOC concentrations. However, this may be because of the source water, which had low DOC and BDOC concentrations (1.2 and 0.11 mg/L, respectively).

Organic matter affects distribution system water quality by generating color, undesired taste, and odors. When chlorine (Cl_2) residual is provided for disinfection, organic compounds are responsible for high Cl_2 demands and the formation of disinfection by-products (DBPs). Also, the biodegradable organic matter (BOM) that is not removed during water treatment can cause bacteria to proliferate along the distribution system. This in turn decreases water quality, accelerates corrosion rates of pipes, and may increase the incidence of bacteriological diseases

(Geldreich, 1996). Since promulgation of the Trihalomethane Rule in 1979 and the Surface Water Treatment Rule in 1989, the goal for water utilities has been to balance microbial disinfection while minimizing DBP formation (Jacangelo et al., 1997). This goal can be achieved by disinfecting water with disinfectants other than free Cl_2, such as ozone (O_3), and by decreasing the Cl_2 residual concentration in the distribution system. Widely used in Europe, O_3 oxidation is a common water treatment process used to remove water pollutants, odors, color, and tastes. O_3 also has strong disinfectant properties. However, ozonation may convert large organic molecules into smaller, more biodegradable organic molecules commonly quantified as assimilable organic carbon (AOC) or biodegradable dissolved organic carbon (BDOC) concentrations. These can serve as “food” sources for bacteria present in the water and pipe biofilms throughout the distribution system, potentially resulting in bacterial regrowth.

The effects of ozonation on AOC and BDOC concentrations in finished waters and the corresponding potential impact on bacterial regrowth in distribution systems have been investigated in several bench-scale and pilot-scale studies conducted under controlled conditions. However, few or no full-scale data are available to quantify short-term as well as long-term changes in AOC and BDOC concentrations under dynamic field conditions, which may be significantly different from bench-scale and pilot-scale conditions. Thus, in this project, the authors monitored water quality and biological conditions in a full-scale distribution system before and after the introduction of O_3.

INTRODUCTION

BOM

To assess the BOM level in water, different bioassay methods were developed. The two most prominent types of bioassays were (1) methods expressing BOM in AOC concentrations and (2) methods expressing BOM in BDOC concentrations.

The BDOC content represents the fraction of dissolved organic carbon (DOC) that has been assimilated and mineralized by a heterotrophic flora (Huck, 1990). It is the fraction of DOC that is biodegraded under the test conditions of long contact times and/or a high concentration of adapted biomass. The inoculum for the test consists of environmental bacteria suspended or fixed on a support, such as sand or porous beads. BDOC is the difference between the initial DOC of the water sample and the minimum DOC observed during the incubation period of 28 days for suspended indigenous bacteria, or 5–7 days for bacteria attached to sand (Volk et al., 1994). BDOC typically comprises 9–30% of the total organic carbon (TOC) (Joret et al., 1991).

AOC refers to the fraction of the TOC that is most readily utilized by bacteria, resulting in a measured increase in biomass concentration. AOC typically comprises just a small fraction (0.1–9.0%) of the TOC. The inoculum can be a mixture of pure bacterial strains cultivated under laboratory conditions (*Pseudomonas fluorescens* P17 and *Spirillum* NOX) (*Standard Methods*, 1995; Volk et al., 1994) or mixtures of environmental bacteria characterized by a great nutritional versatility (Van der Kooij, 1992; Stainfield & Jago, 1987). Bacterial growth is monitored in the water samples by colony counts. The maximum growth ($N_{\max}$) observed during the incubation is converted into AOC by using a growth yield of the bacteria from calibration curves derived from standard concentrations of organic compounds (i.e., acetate and oxalate).

Bacterial Regrowth

The water industry uses the terms “regrowth” and “aftergrowth” synonymously to describe the unexplained occurrence of coliforms in the distribution system. Mechanisms responsible for the presence of coliform bacteria in drinking water supplies are often unknown or poorly understood, so the term “episode” or “occurrence” is more appropriate for describing the presence of coliform bacteria (Van der Kooij, 1992). Increased counts of chemoheterotrophic bacterial colonies in distribution system water, which are more accurately referred to as regrowth (Joret et al., 1991), result from bacterial contamination outside the distribution system or multiplication of bacteria within the system. The major concern involving regrowth is the multiplication of potentially pathogenic (usually opportunistic) bacteria, such as *Legionella pneumophila* (Geldreich, 1996).

Bacterial growth in distribution systems has been related to the following: (1) environmental factors, such as temperature and rainfall; (2) the availability of nutrients; (3) the ineffectiveness of disinfectant residuals; (4) corrosion and sediment accumulation; and (5) hydraulic effects (LeChevallier, 1990).

Water temperature is a rate-limiting parameter in microbial growth. Within the range normally seen in water distribution systems, as the temperature increases, microbial activity and growth rates in the water increase; that is, as the temperature increases from 15°C, microbial activity in the water increases until (for typical flora in distribution systems) temperatures exceeding roughly 35°C are reached.

For bacterial growth to occur, major nutrient sources such as nitrogen, phosphorus, and carbon must be present in the water or released by pipe materials. As a result of the low phosphorus levels found in treated water (<0.5 mg/L), phosphorus is a primary limiting nutrient for bacterial growth (Miettinen et al., 1999; Sathasivan et al., 1997). Nitrogen, on the other hand, is usually readily available as organic nitrogen, ammonia, nitrate, and nitrite, all of which are undesirable in drinking water. Finally, organic carbon is used by heterotrophic bacteria to produce new cellular material (assimilation) and as an energy source (dissimilation). The organic carbon in treated water is composed primarily of humic and fulvic acids, polymeric carbohydrates, proteins, and carboxylic acids. In many water distribution systems, it is assumed that BOM is the limiting nutrient for regrowth (LeChevallier et al., 1991), although a few studies suggest that phosphorus may be limiting for some systems (Miettinen et al., 1999; Sathasivan et al., 1997).

The third factor in bacterial regrowth is the inability to maintain a disinfectant residual. It is necessary to maintain a 3- to 5-mg/L free Cl_2 residual to reduce bacterial biofilms by 3 logs, but a 1-mg/L chloramine residual can reduce viable counts by more than 2 logs for biofilm growth on galvanized, copper, or polyvinyl chloride (PVC) pipe surfaces (LeChevallier, 1990). However, it has been observed that increasing residual Cl_2 doses can increase AOC concentrations because the Cl_2 acts as an oxidant in the breakdown of larger organic carbon molecules into smaller organic carbon compounds (Hambsch & Werner, 1993; LeChevallier et al., 1992; Van der Kooij, 1987). Moreover, Cl_2 residual in the presence of DOC sometimes has the potential of producing DBPs.

Corrosion and sediment accumulation can protect attached bacterial growth. LeChevallier et al. (1996) suggest that disinfection of biofilm bacteria on galvanized iron, copper, or PVC pipes was effective using 1 mg/L of free Cl_2 or monochloramine, but disinfection of organisms on iron pipes was ineffective even when they were exposed to 5 mg/L free Cl_2 for several weeks. In these circumstances, the corrosion products not only provide a habitat for microbial growth but protect the microorganisms from disinfection by reacting with Cl_2-based compounds.

The last factor in bacterial regrowth in the distribution system is hydraulic effects. Low water velocities or dead ends that lead to stagnation of water in the distribution system allow loss of disinfectant residual and subsequent microbial growth (LeChevallier, 1990).

BOM and Bacterial Regrowth

Organic compounds, either dissolved or particulate, contribute to the growth of heterotrophic bacteria, which use organic carbon to produce new cellular materials or to obtain energy. As a result, BOM is gradually consumed as the water travels along the distribution system.

AOC can be used to predict and control regrowth of heterotrophic bacteria. A significant correlation has been observed between the AOC concentration and the density of heterotrophic bacteria in distribution water supplies (Van der Kooij, 1992; LeChevallier et al., 1987). Van der Kooij (1992) showed that heterotrophic bacteria in a nonchlorinated system did not increase when AOC levels were lower than 10 μg/L; LeChevallier (1990) suggested that regrowth may be limited by AOC levels less than 50–100 μg/L. Van der Kooij et al. (1999) typically work with distribution systems that have no disinfectant residuals. LeChevallier (1999), however, typically works with distribution systems that have a disinfectant residual, thus partially explaining this difference in AOC levels associated with biological stability.

Joret et al. (1991) suggested that BDOC values represent 10–30% of the total DOC content of drinking water and that a correlation between nutrient availability and regrowth potential of water exists for P17 and coliforms. Block et al. (1993) recommended an absence of biodegradable organics after water treatment to limit bacterial regrowth. Servais et al. (1993) have associated biological stability, which corresponds to no BDOC consumption within the distribution system, with a BDOC concentration of 0.16 mg/L or less in the finished water. Volk (1994) determined a value of 0.15 mg/L at 20°C and 0.30 mg/L at 15°C for achieving biological stability in distribution systems of the suburbs in Paris, France. Finally, coliform occurrences were related to consumption of BDOC exceeding more than 0.10–0.15 mg/L (Volk & Joret, 1994).

Removal of Organic Material During Water Treatment

Organic matter can be removed by ozonation combined with biological filtration, which removes the biodegradable molecules produced after oxidation. An alternative is membrane filtration. The principal objectives of organic matter reduction are to

- remove taste, odor, or color of the water;
- reduce the formation of chlorination by-products;
- reduce oxidant demand and stabilize the residual disinfectant concentration in the distribution system; and

- prevent bacterial regrowth in the distribution system through the increased biological stability of the treated water.

O_3 Oxidation

O_3 is a strong disinfectant used as an alternative to Cl_2, with the major advantage that no DBPs are produced when it is used. Another advantage is the low contact times (seconds to several minutes) required for O_3 inactivation of microorganisms; characteristically, longer disinfection times are used for Cl_2 or chloramines. The major disadvantage is the lack of a residual after ozonation (Haas, 1990).

O_3 is a colorless gas, produced from the action of electric fields on oxygen. It is highly unstable in the gas phase; in clean vessels at room temperature, its half-life in air is 20–100 hr (Haas, 1990). When it is dissolved in water, O_3 can react with water to form hydroxyl radicals. These radicals cause increased decomposition of O_3 and are responsible for nonselective oxidation of a variety of organic materials. As a result of this decomposition, the half-life of O_3 in water becomes quite short, on the order of seconds to minutes (Haas, 1990).

O_3 is often applied as a pretreatment prior to a biological granular activated carbon filter to improve the removal of organic materials and chemical pollutants (*Ozone in Water Treatment*, 1991). O_3 oxidation can be implemented at the beginning of the treatment chain, after the settling/filtration process in combination with a biological filter, or at the end of the treatment train. When O_3 is used to remove color from water, oxidate mineral compounds, or improve coagulation processes, the applied O_3 doses are low (0.2 mg O_3/mg DOC) and produce a limited amount of nutrient, which is easily removed during later treatment steps. When ozonation is combined with biological filtration to partially oxidize micropollutants for biodegradation, applied ozonation rates of 0.5–1.0 mg O_3/mg DOC correspond to optimal BDOC production. In addition, the use of O_3 alone as a disinfectant (typically 1–7 mg/L O_3) yields significant BDOC production. Consequently, biological stability must be achieved by biological or membrane filtration or by using high doses of an oxidant with a persistent effect, such as Cl_2, chloramine, and chlorine dioxide, to provide a residual (Joret & Prévost, 1996).

O_3 can significantly alter the chemical matrix of carbon compounds present in the water by forming several organic and inorganic by-products, which are formed when O_3 reacts with natural organic matter present in drinking water supplies (Najm & Krasner, 1995). O_3 reacts directly on aromatics, unsaturated organic compounds (e.g., double bonds: $C{=}C$, $C{=}O$, $C{=}S{=}N{-}H$), and amino compounds ($-NH_2$), leading to the formation of saturated polycarbonaceous compounds and methyl ketones, diacids, dialdehydes, aldehydes, and acids (Volk et al., 1997). This leads to a decrease in DOC (because of oxidation to carbon dioxide [CO_2]) and ultraviolet $(UV)_{254}$ (carbon-carbon double bonds are known to absorb UV light at 254 nm) and an increase in AOC and BDOC (Volk et al., 1997; Najm & Krasner, 1995; Hacker et al., 1994; Roche et al., 1994; Volk et al., 1993). The previously noted studies reported an approximate increase in BDOC of 20–30% and an approximate threefold increase in AOC after ozonation. Most of the increase in AOC was a result of increased yields of *Spirillum* strain NOX organism, because this organism has been found to be more suited than P17 to utilization of the oxygenated organic compounds found in ozonated water (Najm & Krasner, 1995).

Cipparone et al. (1997) determined that the maximum colony count of suspended bacteria measured during an incubation period was higher in ozonated samples that were not biodegraded using biological filters than in samples that were biodegraded. The maximum bacterial populations were also higher in ozonated water compared with those in unozonated water. Also, the research found that suspended bacteria did not appear in distribution system water until the free Cl_2 residual disappeared, despite the presence of an attached and acclimated biofilm. The appearance of bacteria was related, therefore, to the length of time the Cl_2 residual could be maintained. Thus, if Cl_2 residuals last longer because of ozonation or biodegradation, the regrowth of bacteria could be delayed even if the original level of AOC was high. Where Cl_2 was added after ozonation to provide a residual, Cipparone et al. (1997) found that bacterial regrowth and Cl_2 demand could be reduced by biological treatment.

METHODS AND MATERIALS

Water Treatment Plant

The full-scale facility in this study was the Pine Hills Water Treatment Plant operated by the Orlando Utilities Commission (OUC). Initially, the Pine Hills plant treated high-quality Floridan aquifer water with chlorination and did not require aeration because the source water had relatively low sulfide levels (average 0.44 mg/L as total sulfide). Other average concentrations for the raw water were typically 97 mg/L as calcium carbonate ($CaCO_3$) alkalinity, 106 mg/L as $CaCO_3$ hardness, 0.23 mg/L ammonia, and 0.50–0.81 mg/L TOC. Treatment for hydrogen sulfide consisted solely of chlorination (2 mg/L as Cl_2); in addition, caustic (pH control), hydrofluorosilicic acid, and 2 mg/L Cl_2 for disinfectant residual were added to the process stream. The Pine Hills plant had a design capacity of 25 mgd (9.46×10^4 m^3/d) and averaged 18.3 mgd (6.93×10^4 m^3/d) during the study.

Water from the Pine Hills plant mixes with water from other adjoining plants in the distribution system, but the mixing zones are at least 3–4 mi (4.8–6.4 km) from the plant. Because the Pine Hills plant is in the northwest corner of the OUC service area, mixing with other plants occurs only to the south and east of the plant. Sampling points were located in the section of the distribution system that contained water from just the Pine Hills plant. After a year of monitoring, OUC installed an O_3 process at the Pine Hills plant (at the end of May 1998) to improve the water's aesthetic quality. The design O_3 dose for the Pine Hills plant was set at 3.1 mg/L; the actual O_3 dose averaged 4.7 mg/L during the study. A significant increase in the levels of AOC entering the distribution system was expected at this high O_3 dose, along with the lack of biological filtration after ozonation. To repress regrowth, 2 mg/L of free Cl_2 for disinfectant residual was added to the process stream after ozonation. The plant had been monitored monthly for one year prior to changing to O_3 (monitoring started on May 28, 1997) and for one year after the change occurred, with weekly monitoring the month prior to and after the change. During the year of ozonation, there were multiple failures of the ozonation system. As a result, the plant had only O_3 online for eight months.

To thoroughly assess biostability, both AOC and BDOC were monitored in the Pine Hills Water Treatment Plant and its distribution system. AOC was measured using the rapid method of LeChevallier et al. (1993), except that plate counts rather than adenosine triphosphate (ATP) fluorescence were used to enumerate bacteria. For BDOC, the authors followed the technique using sand-fixed bacteria (Joret et al., 1991). Initially, an older-model TOC analyzer[*] was used, but this was replaced by a new model[†] for TOC analysis six months into the study.

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[*] TC-180, Dohrmann, Cincinnati, Ohio

[†] Phoenix 8000TM, Dohrmann, Cincinnati, Ohio

Sample Collection, Transportation, and Storage

Samples were collected following *Standard Methods* (1995) procedure 9060A. The procedure requires the addition of a 10% sodium thiosulfate solution to neutralize the Cl_2 residual in the samples. The addition of thiosulfate did not significantly stimulate the growth of P17 or NOX and thus had no effect on AOC concentrations (Escobar & Randall, 1999; Kaplan & Bott, 1989). After collection, samples were transported and stored according to procedure 9060B (*Standard Methods*, 1995).

Residual Cl_2

Free and combined Cl_2 concentrations were measured at the time of sampling using the colorimetric DPD method.

Reagent Water and Glassware

Water used to prepare all solutions was ultrapure water.* The water quality met or exceeded the type I reagent-water specifications provided in Table 1080:I of *Standard Methods* (1995). Purified water used for microbiological testing met the quality criteria specified in Table 9020:I of *Standard Methods* (1995). Laboratory chemicals were American Chemical Society reagent-grade or higher purity. Glassware was cleaned in a sink using a detergent wash, an acid wash, and several distilled water rinses. The glassware was then muffled at 550°C for 4 hr to remove any organic contamination. All glassware used for microbiological testing was autoclaved prior to use. Silicon/polytetrafluoroethylene septa were pretreated by heating in a 100-mg/L sodium persulfate solution for 30 min without allowing it to boil.

Heterotrophic Plate Count (HPC) Analysis

HPCs were performed using the spreadplate method on R2A agar incubated at 25°C for seven days, according to procedure 9215B in *Standard Methods* (1995). All results were expressed in colony-forming units per milliliter. The procedure was performed entirely inside a laminar flow hood. Sample dilutions (usually 1 mL of sample transferred to the plate) were prepared so that the total number of colonies on a plate was less than 300.

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* Quality equivalent to that produced by a Milli Q-UV plus system, Millipore Corp., Bedford, Mass.

AOC Bioassay

AOC was measured using the rapid method of LeChevallier et al. (1993), except that plate counts rather than ATP fluorescence were used to enumerate bacteria, in conjunction with *Standard Methods* (1995) procedure 9217 and the method of Van der Kooij (1992). The procedure used is outlined in detail in Escobar and Randall (1999). Quality control for the AOC bioassay was performed using blank controls, 100-μg/L sodium acetate standards, and duplicate samples. The 100-μg/L sodium acetate standards inoculated with P17 produced an average AOC concentration of 93.80 ± 20.00 μg/L as acetate-C; for NOX, the standards produced an average AOC concentration of 77.20 ± 12.53 μg/L as acetate-C. Experimental yield values from acetate standards for P17 were $4.08 \pm 0.81 \times 10^6$ cfu/μg acetate-C and $9.26 \pm 1.50 \times 10^6$ cfu/μg acetate-C for NOX, which compared reasonably well with the literature values specified in *Standard Methods* (4.1×10^6 and 1.2×10^7 cfu/μg acetate-C for P17 and NOX, respectively). Also, controls were kept to determine air and/or agar contamination by opening plates for 5 minutes under the laminar flow hood, followed by incubation, and by incubating unopened plates. No controls were made to assess the effect of the thiosulfate because it has been determined not to affect AOC concentrations (Kaplan & Bott, 1989) and is included in *Standard Methods* (1995).

BDOC

The procedure for BDOC determination followed the technique using sand-fixed bacteria (Escobar & Randall, 1999; Joret et al., 1991). Minimum DOC values were typically observed on day 4 or 5 for the sample matrix analyzed in this study. The biological sand was washed with organic-free distilled water and stirred using a pretreated glass rod for 15 repetitions twice per week while being stored inside aerated stainless-steel beakers in a tap water with a low organic content (with neutralized Cl_2 residual). Before DOC analysis, samples were withdrawn using muffled glass syringes with muffled double-membrane filters (0.45-μm pore size). The procedure was considered valid only if the BDOC concentration recorded in activity control flasks (with 2 mg/L acetate-C) was 2 ± 0.4 mg/L C.

RESULTS AND DISCUSSION

Before O_3

The Pine Hills Water Treatment Plant was sampled for one year (from May 1997 to May 1998) prior to switching to O_3. Sampling was performed monthly from May 1997 to February 1998 and weekly from March 1998 until the day of the switch to O_3, May 26, 1998. Thus, there were 20 sampling campaigns prior to ozonation. There were seven sampling sites for each sampling campaign—the raw water, the plant finished water, and water in the distribution system (two average hydraulic retention time [HRT] sites [4 and 6.5 hr] and three maximum HRT sites [10.5, 18.5, and 27 hr]). The majority of pipes within the plant as well as in the distribution system were made of ductile iron, iron concrete lined, and PVC. The raw water to the plant was from the deep Floridan aquifer, which displayed very stable and low organic carbon concentrations, averaging 69 ±35.2 µg/L as acetate-C for AOC and 0.11 ± 0.14 mg/L for BDOC. The raw water had an average DOC concentration of 1.20 ± 0.49 mg/L and UV_{254} of 0.0268 ± 0.0024 cm^{-1}. HPCs for the raw water averaged 178 cfu/mL, and 2 mg/L of chlorine, as Cl_2, were added to the treated water after aeration. All statistical analyses are shown in Table 1.

The treated water from the Pine Hills plant displayed average AOC concentrations of 70 ± 24.5 µg/L as acetate-C, BDOC of 0.12 ± 0.05 mg/L, DOC of 1.04 ± 0.31 mg/L, and UV_{254} of 0.0200 ± 0.004 cm^{-1} prior to ozonation. As shown in Figure 1, the average AOC concentrations in the distribution system did not vary significantly, although the average value for the highest HRT site was probably slightly higher than the effluent values. This can happen when more slowly degradable BOM is converted to AOC because of a breakdown of organic carbon compounds in the presence of Cl_2.

Figure 1 also shows the average BDOC concentrations for the same period and sampling sites as AOC. In contrast to AOC (Wilcoxon p-value = 0.813), BDOC increased significantly (Wilcoxon p-value < 0.0001) with respect to the finished water at all distribution system sites. For the average HRT sites, BDOC increased slightly from a finished water value of 0.12–0.18 mg/L for both the 4- and 6.5-hr sites. This could have been due to the use of free Cl_2 for disinfection, because Cl_2 has been observed to partially oxidize DOC (Hambsch & Werner, 1993; LeChevallier et al., 1992; Van der Kooij, 1987) or pipe materials (LeChevallier et al., 1987). The BDOC was even greater at two of the three maximum HRT sites—0.23 mg/L for the 10.5- and 18.5-hr sites. The small decrease to 0.16 mg/L for the 27-hr site could have been from some conversion of BDOC to AOC by microbial activity. This hypothesis is consistent with the high AOC found in this sampling site: 81 µg/L as acetate-C.

Figure 2 shows the average DOC and UV_{254} for the sampling period before introduction of O_3 versus the HRT of the sampling site. DOC was observed to vary slightly during the treatment process or in the distribution system. Figure 2 also shows the average UV_{254} for each sampling site. UV_{254} decreased significantly (Wilcoxon $p\text{-value} < 0.0001$) during the treatment process, probably as a result of oxidation of carbon-carbon double bonds during ozonation. UV_{254} did not change significantly in the distribution system, although the data suggest that UV_{254} might be slightly lower at the three long-term HRT sampling points.

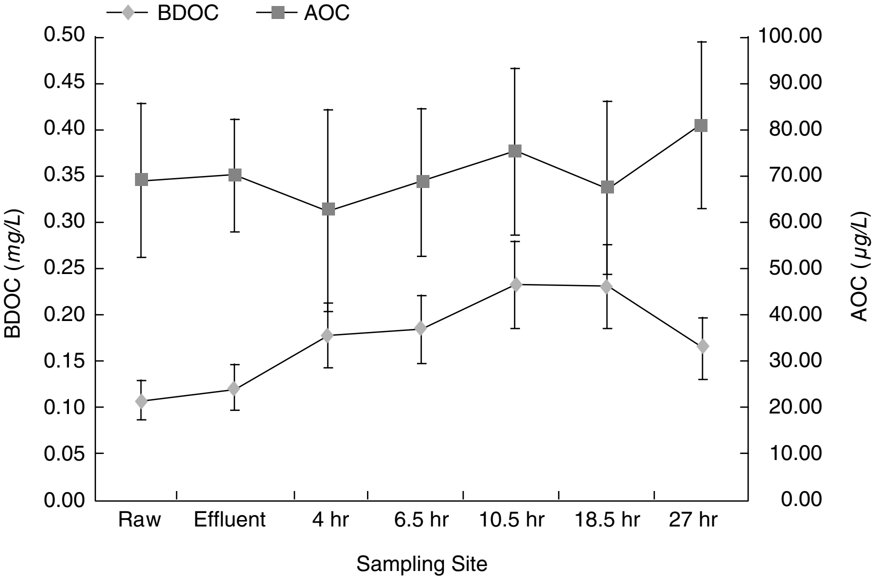
Table 1 Nonparametric Wilcoxon statistical analysis*

	Parameter 1†	Parameter 2	Z	p-value (two-tailed)	Significant Difference
Before ozone	$AOC_{\text{Sampling site}}$	AOC_{Raw}	–0.236	0.813	No
	$BDOC_{\text{Sampling site}}$	$BDOC_{\text{Raw}}$	–4.188	0.000	Yes
After ozone	$AOC\ Raw_{\text{Before}}$	$AOC\ Raw_{\text{After}}$	–1.013	0.311	No
	$AOC_{\text{Sampling site}}$	AOC_{Raw}	–3.180	0.001	Yes
Short-term	$Residual_{\text{Before}}$	$Residual_{\text{After}}$	–0.458	0.647	No
	DOC_{Before}	DOC_{After}	–0.325	0.745	No
	AOC_{Before}	AOC_{After}	–4.029	0.000	Yes
	$BDOC_{\text{Before}}$	$BDOC_{\text{After}}$	–0.104	0.917	Yes
	HPC_{Before}	HPC_{After}	–3.653	0.000	Yes
Long-term	$Residual_{\text{Before}}$	$Residual_{\text{After}}$	–2.750	0.006	Yes
	DOC_{Before}	DOC_{After}	–3.202	0.001	Yes
	AOC_{Before}	AOC_{After}	–5.087	0.000	Yes
	$BDOC_{\text{Before}}$	$BDOC_{\text{After}}$	–0.375	0.707	No
	HPC_{Before}	HPC_{After}	–4.214	0.000	Yes

* Confidence level is 95%.

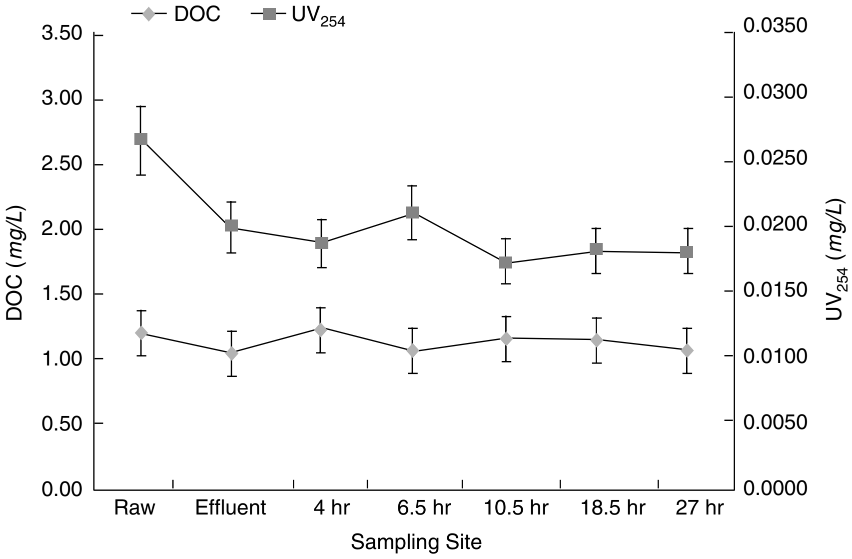
† AOC—assimilable organic carbon, BDOC—biodegradable dissolved organic carbon, DOC—dissolved organic carbon, HPC—heterotrophic plate count

On average, 2 mg/L of free Cl_2 were added for residual disinfectant when the raw water was treated prior to the switch to ozonation. Figure 3 shows the average Cl_2 residual and HPCs (using a logarithmic scale) versus the HRT of the sampling site. The free Cl_2 residual steadily decreased as the retention time of the treated water increased. The average residual was 1.6 mg/L in the plant effluent, 1.1 mg/L for both average HRT sites, and 0.9 mg/L at the 6.5- and 10.5-hr sites. At the 27-hr site, the residual was 0.6 mg/L. This significant decrease in the Cl_2 residual probably occurred because the temperature of the distribution system water was high, ranging from 17° to 29°C, as well as because of the reactivity of the free Cl_2 with pipe materials and the presence of reduced compounds in solution and of any biofilms. The HPCs increased slightly at the maximum HRT sites compared with the average HRT sites, which corresponded to the decreased Cl_2 residual.



AOC—assimilable organic carbon, BDOC—biodegradable dissolved organic carbon. Whiskers represent one standard deviation.

Figure 1 AOC and BDOC versus sampling site HRT (average of 20 sampling campaigns) prior to ozonation

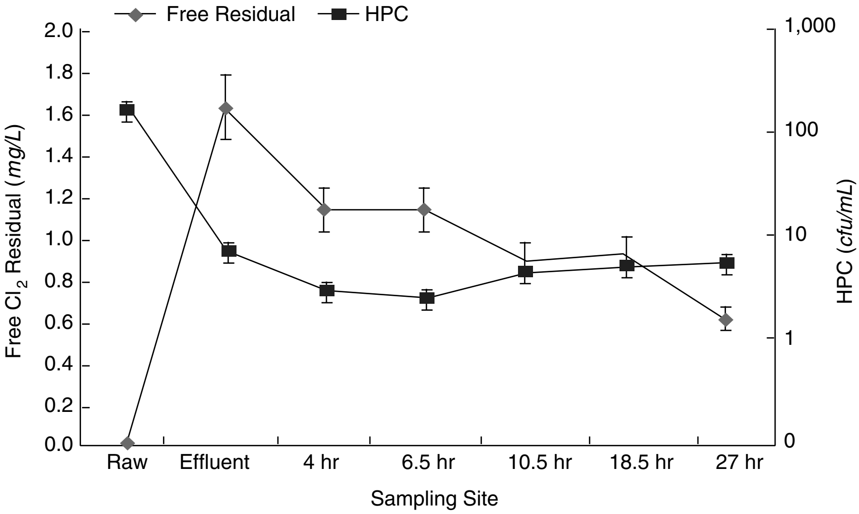


DOC—dissolved organic carbon, UV_{254}—ultraviolet absorbance at 254 nm. Whiskers represent one standard deviation.

Figure 2 DOC and UV_{254} versus sampling site HRT (average of 20 sampling campaigns) prior to ozonation

After Switching to O_3

The ozonators were started on May 26, 1998, but several electrical and mechanical problems occurred during the first eight months of operation, causing the plant to shut down several times. However, only values from sampling campaigns conducted when the O_3 was on-line were used to calculate the data averages presented here. Sampling of the Pine Hills plant, which started on May 27, 1998, did not accurately reflect post-ozone values. Sampling continued weekly until July 15, 1998, when monthly sampling campaigns were resumed until May 1999, one year after the start of ozonation. On June 10, 1998, the primary raw water well (well 5: DOC = 1.1 mg/L, BDOC = 0.11 mg/L, and AOC = 88 µg/L as acetate-C) that fed the plant had pump problems and was shut down. It was replaced by well 1 (DOC = 0.9 mg/L, BDOC = below method detection limit (MDL) of 0.1 mg/L, AOC = 160 µg/L as acetate-C) temporarily. The average AOC concentration (160 µg/L as acetate-C) was indistinguishable from the BDOC concentration (0.10 mg/L or 100 µg/L)



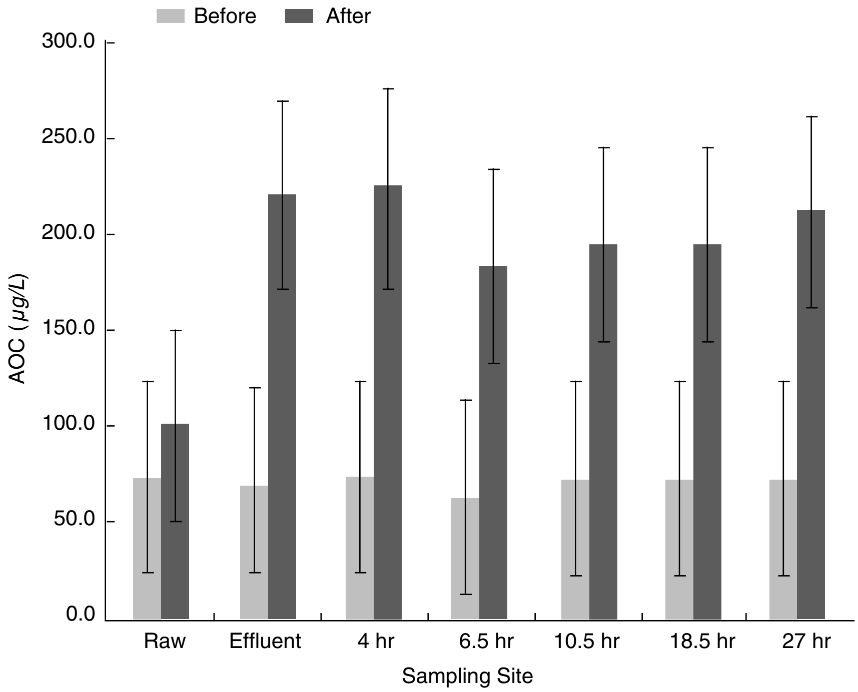
Cl_2—chlorine, HPC—heterotrophic plate count. Whiskers represent one standard deviation; HPC shown with logarithmic scale.

Figure 3 Free chlorine residual and HPCs versus sampling site HRT (average of 20 sampling campaigns) prior to ozonation

because the BDOC MDL is not able to distinguish values less than 0.1 mg/L. Well 5 was returned to use July 15, 1998.

Short-Term Effects of Switching to O_3

The average short-term O_3 dose was 5.8 mg/L O_3. To evaluate the short-term effects of changing from aeration and chlorination to ozonation and chlorination, a weekly sampling period was used. The weekly sampling data before O_3 included 10 sampling points; the weekly sampling data after O_3 included 4 sampling points before O_3 went off-line because of technical and mechanical problems at the Pine Hills plant. Figure 4 shows the short-term effects of ozonation on AOC using only the sampling points that were indicative of O_3 effects. The first observation from this figure was that there was no significant difference in AOC concentration (Wilcoxon p-value = 0.311) in the raw water because it is not affected by the addition of O_3. After the change to O_3, all AOC concentrations in the plant effluent and distribution system were higher than before the change by an approximate average of 130 µg/L as acetate-C. Before O_3, the weekly AOC concentrations ranged from 62 to 73 µg/L as acetate-C for all sampling sites; after O_3, these concentrations were between 182 and

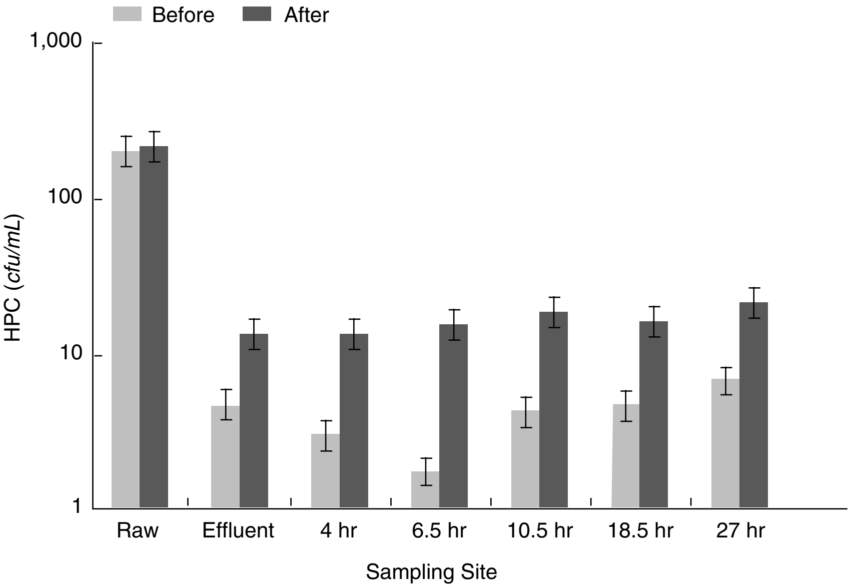


AOC—assimilable organic carbon. Weekly sampling period from March to July 1998; sampling campaigns = 10 before/4 after; whiskers represent one standard deviation.

Figure 4 Short-term effects of ozone introduction on AOC concentrations

224 μg/L as acetate-C. Considering process data only (i.e., raw water and plant effluent), the AOC concentration after O_3 increased by more than 200%, which agrees with previous work performed by Paode et al. (1997), Hacker et al. (1994), and Volk et al. (1993), among others. Thus, as expected, the AOC concentration of the treated water significantly increased (Wilcoxon p-value < 0.0001) from ozonation, probably because of the partial oxidation of larger organic carbon compounds to carboxylic acids facilitated by O_3 oxidation (Volk et al., 1997; Hacker et al., 1994; Roche et al., 1994; Volk et al., 1993).

As shown in Figure 5, the HPCs after the switch to O_3 increased significantly (Wilcoxon p-value < 0.0001). The significant increase in plant effluent and distribution system AOC probably caused the bacterial population in the treated water to increase. Before the change to O_3, the short-term weekly HPC of the treated and distribution system waters

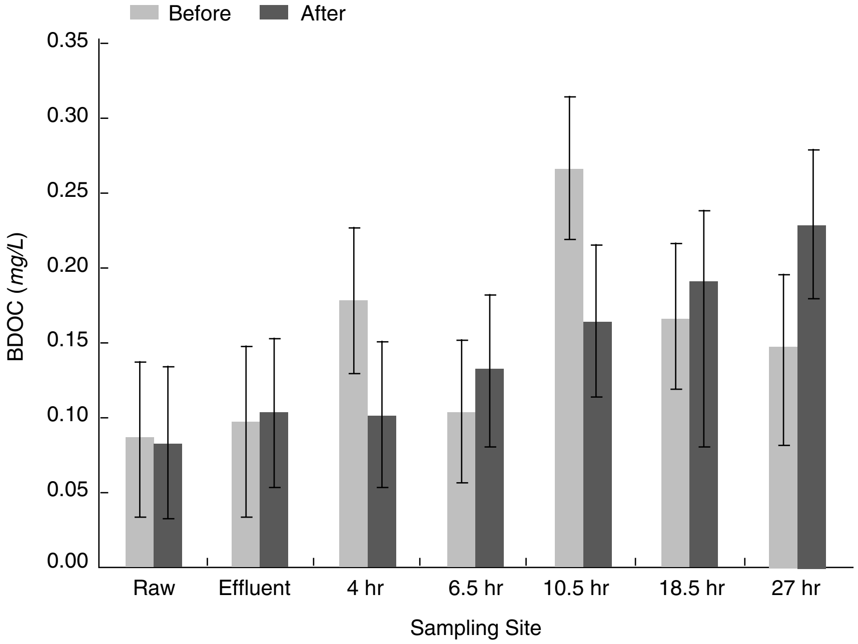


HPC—heterotrophic plate count. Weekly sampling period from March to July 1998; sampling campaigns = 10 before/4 after; whiskers represent one standard deviation; HPC shown with logarithmic scale.

Figure 5 Short-term effects of ozone introduction on HPC concentrations

ranged from 2 to 6 cfu/mL, whereas after ozonation the weekly HPC increased to 13–21 cfu/mL. HPCs observed were very low, probably because of the effectiveness of the disinfectant residual as well as the short hydraulic residence times (4–27 hr) of the distribution system. On average, this was an increase of approximately 12 cfu/mL, or triple the pre-ozone bacterial population in the water. Even though the HPCs tripled, the Cl_2 residual was effective in repressing regrowth. This shows increased potential for biological instability with ozonation if other aspects of system control, such as disinfectant residual and corrosion control, are not maintained diligently.

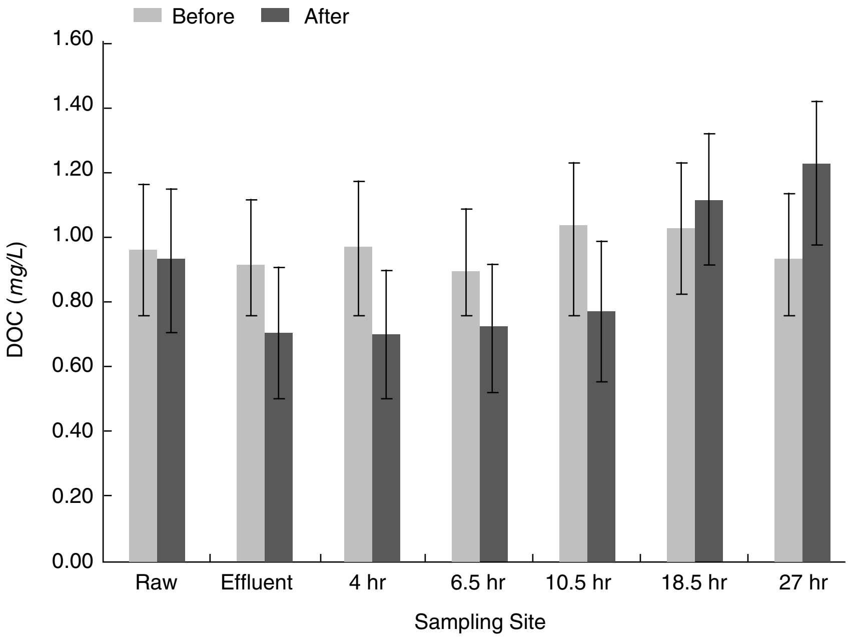
As shown in Figure 6, it was determined that a switch to ozonation caused no significant short-term effects on the BDOC of the treated water, which could be explained by the low BDOC concentration (most < 0.25 mg/L) and initial DOC concentration (< 1.20 mg/L on average) initially found in the water. As observed in Figure 6, the whiskers represent one standard deviation from the BDOC averages. Because the



BDOC—biodegradable dissolved organic carbon. Weekly sampling period from March to July 1998; sampling campaigns = 10 before/4 after; whiskers represent one standard deviation.

Figure 6 Short-term effects of ozone introduction on BDOC concentrations

whiskers overlap, there was probably no significant difference (Wilcoxon p-value = 0.917) in the distribution system BDOC concentrations caused by O_3. However, the BDOC of the plant effluent showed a slight increase of approximately 10% during this transition period, so it is conceivable that more significant changes would be seen in a raw water with higher DOC. DOC concentrations are shown in Figure 7. A switch to O_3 did not significantly affect (Wilcoxon p-value = 0.745) the DOC concentrations of the treated water, as seen by the overlapping whiskers in Figure 7, which represent one standard deviation from the mean values over the sampling period. Even though it was arguably not a significant decrease, the DOC concentration of the plant effluent decreased by approximately 23%. This agreed with previous studies (Paode et al., 1997; Hacker et al., 1994; Volk et al., 1993) because O_3 facilitates the breakdown of high-molecular-weight DOC-forming compounds into low-molecular-weight organic compounds and the formation of CO_2.

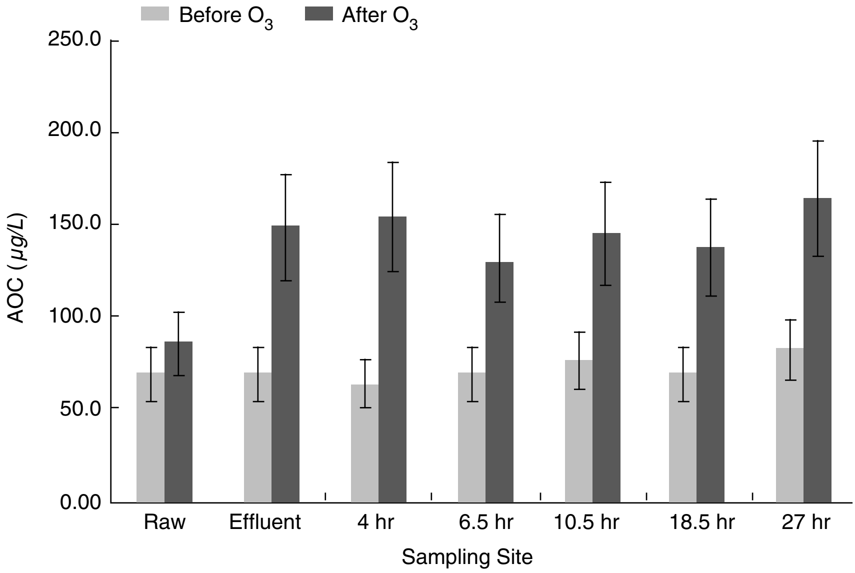


DOC—dissolved organic carbon. Weekly sampling period from March to July 1998; sampling campaigns = 10 before/4 after; whiskers represent one standard deviation.

Figure 7 Short-term effects of ozone introduction on DOC concentrations

Long-Term Effects of Switching to O_3

O_3 levels deviated from the target of 3.1 mg/L to an average of 4.7 mg/L during the entire study period. The long-term data set, which covers the period before ozonation, included monthly sampling campaigns from May 1997 through May 1998. The after-ozonation data set included the sampling campaigns performed in June, July, August, October, and December 1998 and March through May 1999. During these campaigns, O_3 had been on-line for at least five days. Figure 8 shows AOC concentrations before and after ozonation. From Figure 8, it was observed that the average AOC concentration after the switch to O_3 was significantly higher than before O_3 (Wilcoxon p-value < 0.0001). Before O_3, AOC concentrations ranged from 62 to 81 µg/L as acetate-C; after O_3, concentrations ranged from 128 to 162 µg/L as acetate-C. The plant effluent AOC increased by 112% after ozonation compared with that before ozonation.

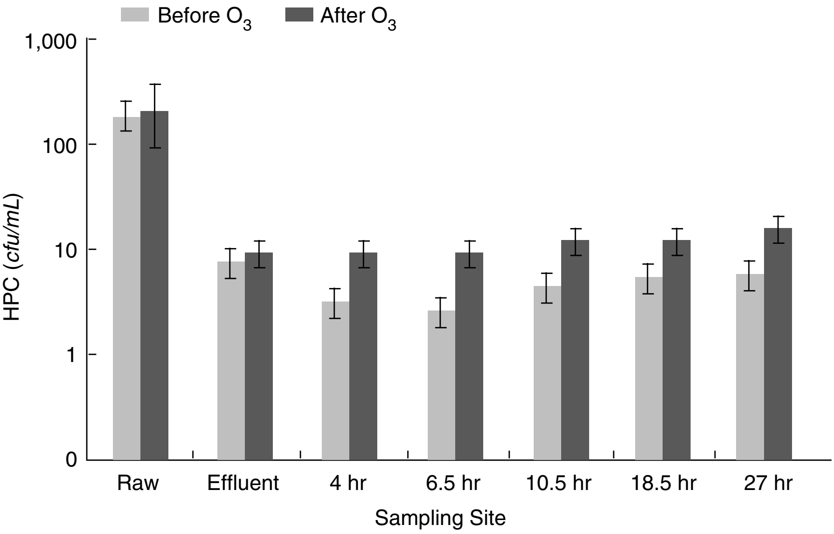


AOC—assimilable organic carbon, O_3—ozone. Monthly sampling period from May 1997 to May 1999; sampling campaigns = 13 before/8 after; whiskers represent one standard deviation.

Figure 8 Long-term effects of O_3 introduction on AOC concentrations

Bacterial counts measured via HPCs before and after ozonation are shown in Figure 9. The raw water averaged 192 cfu/mL during the entire sampling period. Before O_3, the plant effluent and distribution system showed HPCs ranging from 3 to 7 cfu/mL; after O_3, counts ranged from 8 to 15 cfu/mL. These low HPCs were due to large additions of free Cl_2 (2 mg/L) to the effluent water. Figure 9 shows that bacterial counts increased significantly (an increase of 132%) at the distribution system sampling sites after O_3 use compared with the same sites before O_3 (Wilcoxon p-value < 0.0001).

Figure 10 shows BDOC concentrations before and after O_3. BDOC concentrations in the raw water were very low, averaging 0.13 mg/L during the entire sampling period. Before O_3, the plant effluent BDOC was 0.13 mg/L, not significantly different from the raw water, and the distribution system BDOC ranged from 0.16 to 0.23 mg/L. After O_3 usage, the BDOC concentrations of the plant effluent increased to 0.19 mg/L, 44% over raw water values versus 13% without O_3, whereas those of the distribution system increased to 0.19–0.40 mg/L. However,

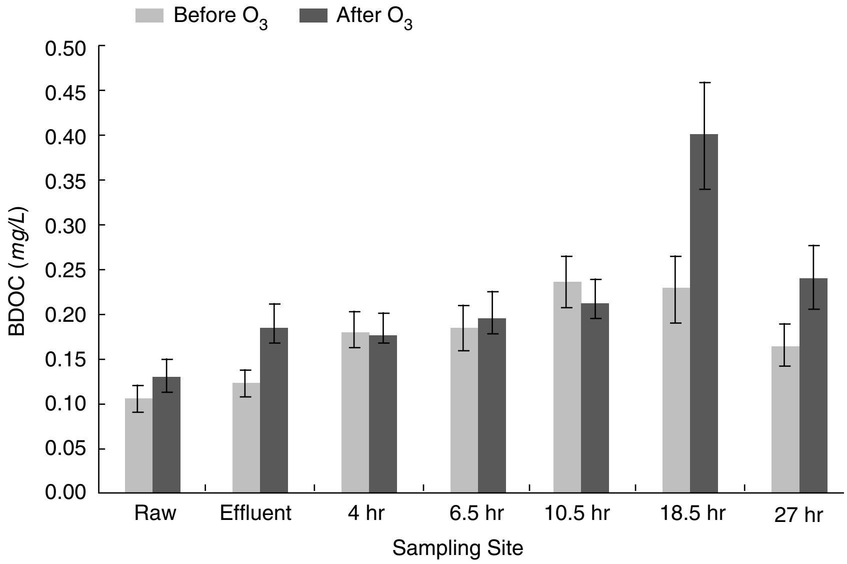


HPC—heterotrophic plate count, O_3—ozone. Monthly sampling period from May 1997 to May 1999; sampling campaigns = 13 before/8 after; whiskers represent one standard deviation; HPC shown with logarithmic scale.

Figure 9 Long-term effects of O_3 introduction on HPC concentrations

taking into account the variability in the data (whiskers represent one standard deviation in Figure 10), an increase in BDOC concentration before O_3 versus after O_3 was observed only in the plant effluent (54% increase from before), the 18.5-hr sampling site (76% increase), and the 27-hr sampling site (48% increase). The fact that an increase in BDOC concentration was not observed at all sampling sites was probably because of the low initial BDOC concentrations, with the small change probably being obscured within the distribution system. The higher BDOC values after ozonation at the long HRT sites might be explained by increased hydrolysis and/or a breakdown of DOC compounds in the presence of Cl_2.

Figure 11 shows the long-term comparison for DOC before and after O_3 usage. The raw water DOC averaged 1.10 mg/L over the two-year sampling period. The plant effluent DOC concentration decreased by 20% after O_3 compared with before O_3. This was expected because O_3 causes DOC compounds to break down into lower-molecular-weight compounds with partial loss as CO_2. Even though Figure 11 shows overlapping standard deviations in DOC values, the decrease in DOC was



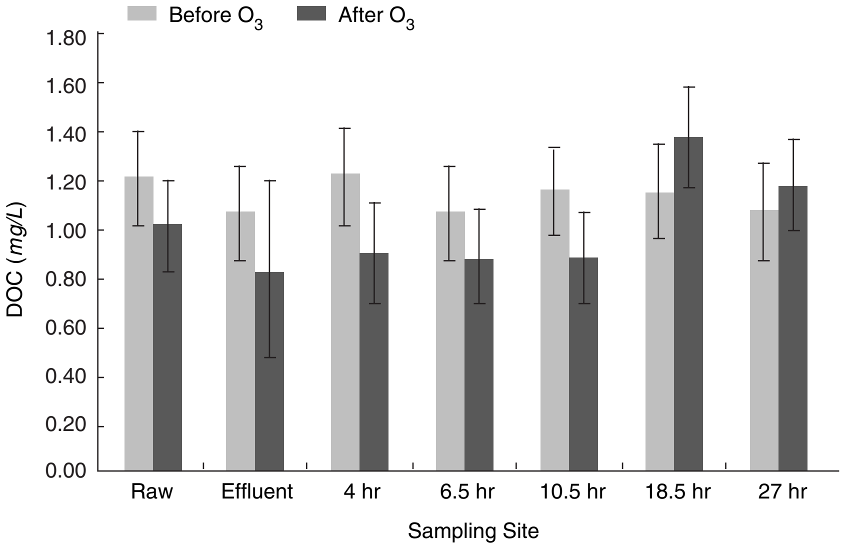
BDOC—biodegradable dissolved organic carbon, O_3—ozone. Monthly sampling period from May 1997 to May 1999; sampling campaigns = 13 before/8 after; whiskers represent one standard deviation.

Figure 10 Long-term effects of O_3 introduction on BDOC concentrations

statistically significant, with a Wilcoxon p-value of 0.001. All distribution system sampling sites displayed small, insignificant decreases in DOC, except for the two highest HRT sites (Figure 11). Figure 12 shows the Cl_2 residual before and after O_3 introduction. The Cl_2 dose and residual at the plant effluent remained constant at 2.0 and 1.6 mg/L, respectively. On the other hand, it increased from an average of 0.9 mg/L in the distribution system before O_3 to an average of 1.1 mg/L after O_3. This was a significant difference, with a Wilcoxon p-value of 0.006.

SUMMARY AND CONCLUSIONS

The Pine Hills Water Treatment Plant, operated by OUC, was sampled for two years—one year before switching to O_3 and one year after the switch. High-quality raw water from the deep Floridan aquifer was treated for several years using aeration and chlorination before switching to ozonation in May 1998. During the year before ozonation was implemented, the AOC concentration of the raw water, plant effluent,

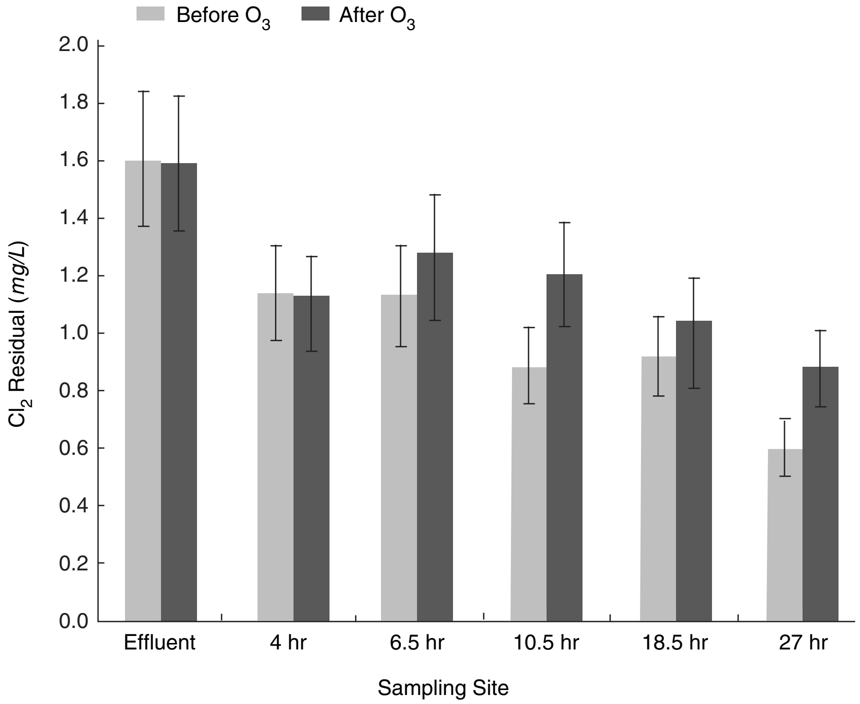


DOC—dissolved organic carbon, O_3—ozone. Monthly sampling period from May 1997 to May 1999; sampling campaigns = 13 before/8 after; whiskers represent one standard deviation.

Figure 11 Long-term effects of O_3 introduction on DOC concentrations

and distribution system averaged approximately 69, 70, and 62–81 μg/L acetate-C, respectively. During the short period after ozonation was implemented (approximately one month of weekly sampling), the AOC of the plant effluent increased to 220 μg/L acetate-C (an increase of more than 200%), whereas the AOC of the distribution system increased to 182–224 μg/L acetate-C (a 200% increase). This was probably because of the breakdown of larger-molecular-weight compounds into smaller compounds measurable as AOC. In the long term (one year) after the switch to O_3, the AOC concentration of the plant effluent stabilized at 148 μg/L acetate-C (a 112% increase from before ozonation), whereas the distribution system AOC concentration ranged from 128 to 162 μg/L acetate-C (an increase of approximately 105% compared with the year before ozonation was implemented).

HPCs tripled in the short term after O_3 implementation; in the long term, the average bacterial count doubled. This increase in HPCs correlated with the significant increase in AOC concentrations following ozonation. The Cl_2 disinfectant residual was very effective in preventing regrowth and maintaining HPCs at low levels.



Cl_2—chlorine, O_3—ozone. Monthly sampling period from May 1997 to May 1999; sampling campaigns = 13 before/8 after; whiskers represent one standard deviation.

Figure 12 Long-term effects of O_3 introduction on Cl_2 disinfectant residual

However, the difference in BDOC and DOC concentrations before and after O_3 was not nearly as significant as the observed changes in AOC concentrations. In the short term, after the switch to ozonation, BDOC concentrations did not show a significant difference from concentrations preceding O_3 use, probably because of the low levels of BDOC in the raw water (0.12 mg/L). In the long term, higher BDOC concentrations as a result of ozonation were observed only in the plant effluent (54% increase), the 18.5-hr sampling site (76% increase), and the 27-hr sampling site (48%), whereas the other sampling sites showed insignificant increases in BDOC concentrations compared with the year prior to ozonation. With the exception of the two sampling sites with the highest HRT (18.5 and 27 hr), the plant effluent and the other sampling sites showed decreases in DOC concentrations after O_3 compared with concentrations before O_3 for both the short term and the long term.

An evaluation of the system before and after O_3 showed that even though the AOC increased significantly after the introduction of O_3, the distribution system regrowth remained low because the Cl_2 addition was high (2 mg/L) and repressed bacterial growth. The plant monitored DBPs and observed little difference before and after the introduction of O_3. The plant had no DBP violations during the two-year study period, and no coliform occurrences were observed. After O_3, the aesthetic quality of the water (i.e., taste and odor) improved, which was a primary objective of O_3 introduction.

Following are the conclusions from this study:

- O_3 use significantly increased AOC concentrations of the plant effluent as well as the distribution system.
- A significant increase in bacterial counts was observed, which correlated with the increase in system AOC levels. However, Cl_2 effectively repressed regrowth before and after O_3.
- Because of the breakdown of large-molecular-weight compounds into AOC compounds and the observed increase in the finished water AOC concentrations, there is increased potential for bacterial proliferation when O_3 is used if a disinfectant residual is not diligently maintained in the distribution system to repress regrowth.
- The use of O_3 caused an overall increase in BDOC, but the increase was not as significant as that for AOC. This may have been because of the very low DOC and BDOC concentrations of the raw water.
- The decrease in DOC concentrations was not significant, probably because of the low DOC concentrations of the raw water (1.10 mg/L).

ACKNOWLEDGMENTS

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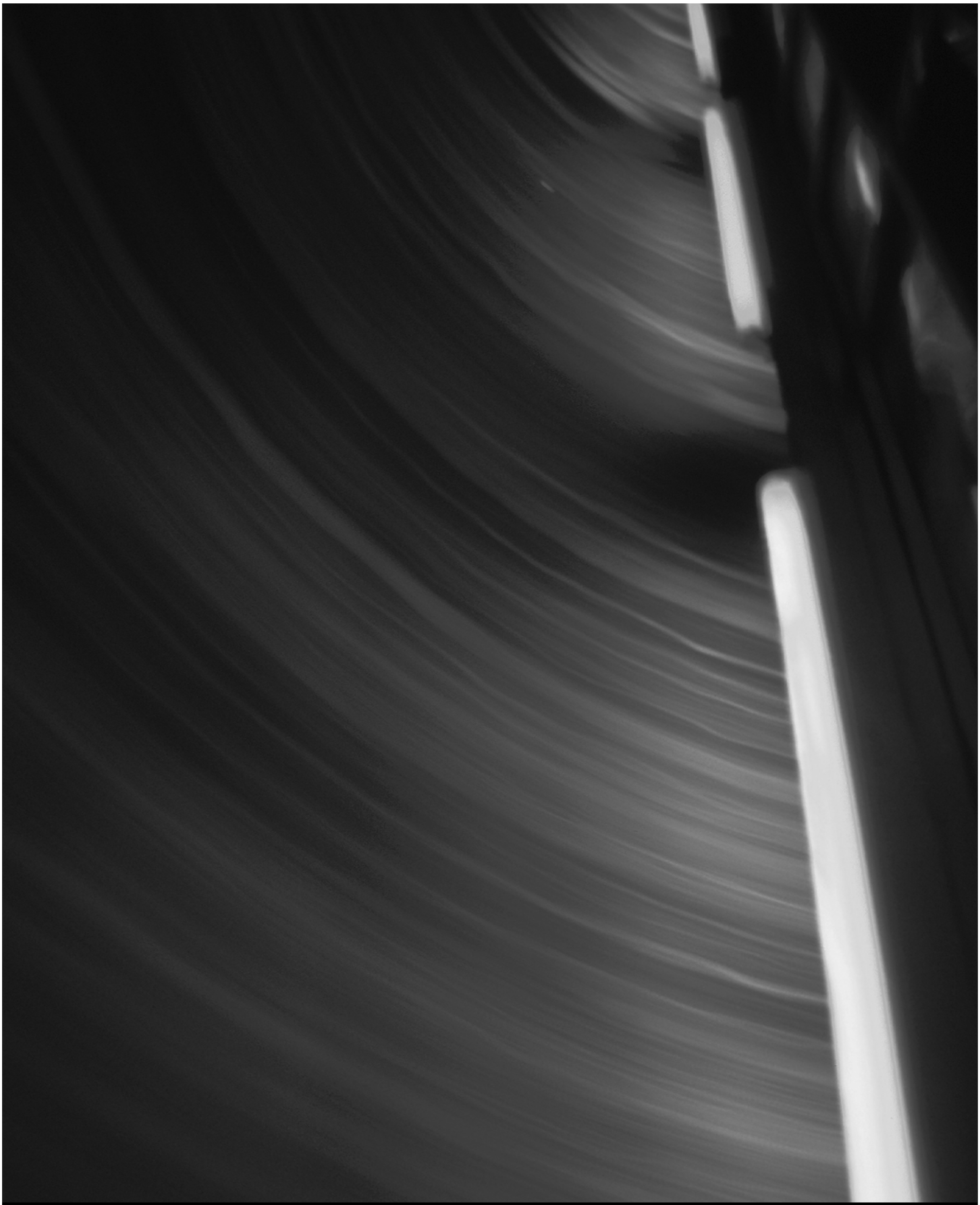

Part 3

Chemical and Physical Issues

CHAPTER · 9

Changes in Chlorine and DOX Concentrations in Distribution Systems

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BACKGROUND

The fate of disinfectants and disinfection by-products (DBPs) has been studied in laboratories under a variety of controlled, constant conditions, but limited information is available about their fate in full-scale distribution systems. A study of the city of Laval (Québec) distribution system examined changes in the concentrations of chlorine and dissolved organic halogens (DOX) as the water flows through the distribution system. Water was collected in small- and large-diameter pipes with respect to the residence time (RT) of each sampling point. Free and total chlorine concentrations decreased with increasing RT in the distribution system. The influence of pipes on chlorine decay was particularly important in warm water; at similar RTs, chlorine concentrations were lowest in the small-diameter pipes, greater in the main pipes, and greatest in the batch incubations. The presence of corrosion by-products in pipes appeared to be the major factor influencing chlorine decay in the distribution system. DOX concentrations generally increased with increasing RT in the presence of free chlorine and decreased when free chlorine concentrations were below the method detection limit. Results point to significant pipe influence on DOX concentrations. DOX decay seemed to be attributable to the chemical instability of some DBPs and the presence of microbial biomass.

INTRODUCTION

As drinking water flows through the distribution system from the treatment plant to the consumer tap, its aesthetic, microbial, and chemical qualities may deteriorate significantly. Increases in bacterial counts, decreases in disinfectant residuals, and changes in taste and odor characteristics are often related to distance from the treatment plant or increase in water residence time (RT). Traditionally, a residual concentration of chlorine—the most commonly used disinfectant—is used to limit bacterial growth or regrowth in the distribution system. However, chlorine dose must be limited because it reacts with organic matter to form halogenated disinfection by-products (DBPs), some of which are of concern because of potential health risks to consumers (USEPA, 1994; NCI, 1976).

Treatment processes, disinfecting conditions at treatment plants, and varying distribution system characteristics influence the residual disinfectant stability as well as DBP formation or decay. Decreases in free and total chlorine concentrations through the distribution system have been observed (Meyer et al., 1993; Capellier et al., 1992; Mathieu et al., 1992; Colbourne et al., 1991). Desjardins et al. (1991) showed that the chlorine decay measured in the distribution system was higher than in laboratory batch incubations. Chen and Weisel (1998) observed that free and total chlorine concentrations were depleted more slowly in cold water than in warm water.

The change in DBPs in distribution systems can be presented in terms of DBP quantity or speciation. DBP formation and change in speciation have been studied in laboratories under a variety of controlled, constant conditions. In distribution systems, however, conditions vary with time and location, influencing both the stability of the DBPs already formed and possibly the formation of additional DBPs. Limited information is available on the change in dissolved organic halogen (DOX) concentration. However, some chlorination by-products, e.g., trihalomethanes (THMs) and haloacetic acids (HAAs), have been studied in distribution systems.

Previous Studies

Increases in THM concentration with increasing RT in the distribution system have been measured (LeBel et al., 1995; Koch et al., 1991; Levine et al., 1987; Brett & Calverley, 1979; Harms & Loogenga, 1977). A 1998 study focused on a full-scale distribution system fed with chloraminated water and a free chlorine residual of 0.5 mg/L Cl_2 (ammonia was added to convert all but approximately 0.5 mg/L of the free chlorine to chloramines); measurements showed increases in THMs with increasing water RT but decreases in HAAs (Chen & Weisel, 1998). These changes occurred faster (closer to the treatment plant) in warm water than in colder water. In a pilot distribution system simulating a maximum RT of three days, increases in THM levels were observed with increasing RT when chlorine was not a limiting factor (Meyer et al., 1993). In the absence of a measurable chlorine residual, THM concentrations stabilized or increased more slowly. When the pilot distribution system was compared with batch incubations, the researchers measured lower THM concentrations in the batch incubations than in the pilot distribution system, which suggested potential interactions with the biofilm and/or that agitation occurring inside the pipes can degrade the THMs (Meyer et al., 1993).

The biodegradability of several chlorinated and brominated compounds has been addressed in the literature. Lévi et al. (1993) conducted experiments in which DBPs were incubated in sterile flasks (samples were filtered) and in nonsterile flasks. Results showed that dichloro-, bromo-, chloro-, and dibromo-acetonitrile, as well as 1,1-dichloro-propanone and chloropicrin were biodegradable. Trichloroacetonitrile and 1,1,1-trichloropropanone were chemically unstable as their concentrations decreased, even in sterile water. According to Meyer (1992), the presence of fixed biomass on pipe walls was responsible for the DBP decay with increasing RT, as statistical analyses showed good correlation between bacterial counts and DBP concentrations.

In another study, a 50% decrease in HAA concentration was measured between the beginning and the end of the distribution system and was attributed to biodegradation, chemical instability, or adsorption (Jammes et al., 1994). When HAA concentrations were measured at the beginning, midpoint, and end of the distribution system, lower concentrations were found at the system's end (Chen & Weisel, 1998; LeBel et al., 1995; Williams et al., 1995; Williams et al., 1994). Low levels of residual chlorine and high heterotrophic plate counts (HPCs) were also recorded at the endpoint of the system, indicating biodegradation of the HAAs, with dichloroacetic acid being more affected than trichloroacetic acid. Specific bacteria capable of HAA degradation were isolated from a distribution system biofilm (Williams et al., 1996; Williams et al., 1995).

Models for DBP Characterization

A variety of models characterizing chlorine decay or change in DBP levels in distribution systems have been proposed (Clark & Sivaganesan, 1998;

Vasconcelos et al., 1997; Biswas et al., 1993; Lyn & Taylor, 1993; Amy et al., 1987). These models take into account several phenomena responsible for the change in chlorine or DBP levels—interaction with organic and inorganic chemicals, reaction with biofilm and/or corrosion by-products, and mass transport between liquid phase and pipe walls. These phenomena were evaluated in the laboratory in constant, controlled conditions, and the kinetics measured were incorporated in the models. As a result, these models are not directly applicable to full-scale distribution systems because they do not consider all of the factors influencing the physical, chemical, and microbial water quality within the distribution system, as well as the dynamic nature of these systems.

Study Objectives

The aim of this research was to determine the dynamics of chlorine and the summation of the halogenated DBPs represented by DOX in the distribution system of the city of Laval, Québec. Specific objectives were to (1) investigate the fate of chlorine and DOX levels in distribution systems; (2) evaluate the effects of several factors (e.g., pipe diameter and materials, biofilm, corrosion by-products, and temperature) that can significantly influence chlorine and DOX concentrations in distribution systems; and (3) elaborate explanatory models that describe the change in chlorine and DOX concentrations in distribution systems and the parameters controlling them.

MATERIALS AND METHODS

The drinking water distributed by the city of Laval is produced by three treatment plants—Chomedey with a capacity of 180,000 m^3/d (47.5 mgd), Pont-Viau (135,000 m^3/d [35.6 mgd]), and St. Rose (110,000 m^3/d [29.0 mgd]). Only Chomedey and Pont-Viau were included in this study because they used chlorine in postdisinfection, whereas St. Rose used chlorine dioxide.

The treatment trains of the Pont-Viau and Chomedey plants were similar: coagulation with alum, flocculation, sedimentation, dual-media filtration on anthracite/sand (10 m/hr [4.1 gpm/sq ft]), ozonation (residual of 0.4 mg/L O_3 after 4 min), and postdisinfection with chlorine. Although these plants feed a unique distribution system, each plant has its own distinctive pressure zone. The city of Laval's topography and infrastructure are such that reservoirs are not required. Booster disinfection was not practiced during samplings.

Sampling

The sampling approach consisted of collecting water as it flowed through the distribution system with respect to the sampling points' RT from the treatment plant. Sampling locations were carefully selected in order to obtain reliable results. Average RT and pressure zones were obtained by hydraulic simulations using two software programs*† that represented the distribution system by links (sections of pipe) and nodes (i.e., intersection of pipes, change in direction, presence of hydraulic equipment such as valves). Distribution system information required by the software programs included diameter and length of pipe sections, Hazen–Williams coefficients, coordinates and elevation of nodes, type of nodes and links, number and type of buildings at each node, average daily consumption at each node, water flow and pressure entering the distribution system, and status of valves. Average values of pressure, flow intensity and direction, and velocity at each node and pipe section were calculated by the modified Newton algorithm with one software program* and by the Hamam algorithm with the other software† (Charron & Jarrige, 1993). With both models, head losses were calculated using the Hazen–Williams or Darcy–Weisbach (simplified or exact) formulas. Average RT at each node was determined by considering various routes followed by the water to reach each sampling location.

Tracer studies have shown differences between real and calculated RTs at some sampling locations. However, control samples collected at two locations (RT of approximately 6 and 12 hr) over a period of 6–7 hr showed only slight variation of water quality over time. The maximum ranges of free and total chlorine residuals were 0.04 mg/L Cl_2, 0.3°C for temperature, 0.4 for pH, 0.22 mg/L C for total organic carbon (TOC), and 0.86 log for HPC bacteria. These results suggested that variations in RT calculations (within 6 hr) did not result in important variations of most parameters measured in the study.

Previous research has shown that pipe diameter influences water quality in distribution systems (Servais et al., 1992; Desjardins et al., 1991). Therefore, the current study considered two different sizes of pipe: main (large-diameter) pipes (200–1,220 mm [8–48 in.]) and small-diameter pipes (150 mm [6 in.]). In order to limit the influence of different pipe materials, sampling routes were targeted to pipes made of

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[*] Rincad, Cedeger Ltd., City of Laval, Québec, Canada

[†] Piccolo, Safège, Nanterre, France

the same material: steel-reinforced concrete for the main pipes (a few sections were in unlined gray iron or ductile cast iron) and unlined gray iron for the small-diameter pipes. The software programs were used to determine the different routes followed by the water to reach each sampling location. Only locations that showed one or a few different routes with a negligible difference in RT were selected. In order to eliminate the influence of service lines (Prévost et al., 1997), samples were collected directly from the pipes using a cleaned, disinfected PTFE tube attached to the end of a perforated stainless-steel (SS 316) tube inserted across the pipes.

Laboratory batch incubations were performed to distinguish the influence of the RT from any potential effect of the pipes themselves. Batch incubations allow differentiation of the effect of constituents within the water from the physical, chemical, and biological nature of the pipes (e.g., diameter, material, possible corrosion by-products, and biofilm). Samples were collected at the treatment plant effluents and incubated in bottles in the dark at the distribution system temperature. Batches were analyzed after specific incubation periods.

Corroborative experiments were conducted using an annular reactor (Figure 1). This device consisted of a cylinder rotating inside a stationary outer cylinder. Four draft tubes drilled into the rotor enhanced water mixing. The annular reactor used was made of polycarbonate, eliminating the potential influence of corrosion. The RT inside the bioreactor (5 hr) was determined by the influent flow (2.3 mL/min) and the useful volume (692 mL); hydraulic conditions were a function of the rotor rotational speed (40 rpm representing a water velocity of < 0.3 m/min [1 ft/min] in a 100-mm [4-in.] diameter pipe). The annular reactor was operated at 20°C and had been in operation for two months when the experiments were conducted.

Analytical Methods

Chlorine concentrations were measured in triplicate by the DPD colorimetric method 4500-Cl G (*Standard Methods*, 1995). Method accuracy was 0.01 mg/L Cl_2 for the laboratory spectrophotometer[*] and 0.03 mg/L Cl_2 for the portable apparatus[†] used in the field. The method detection

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[*] Milton Roy Spectronic 21, Fisher Scientific Co., Montréal, Québec

[†] Milton Roy Mini-Spectronic 20, Fisher Scientific Co., Montréal, Québec

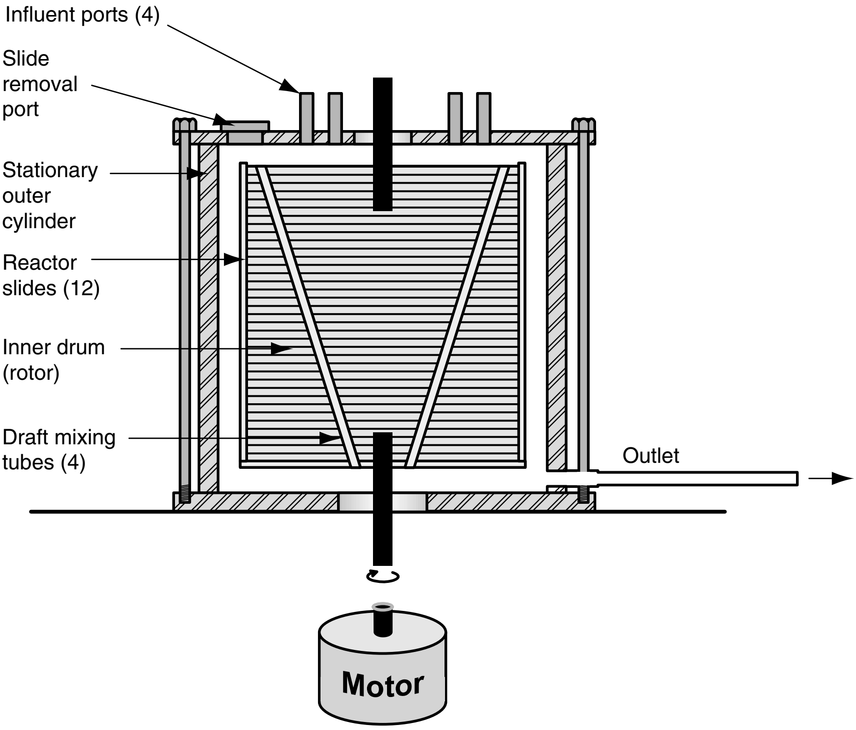

Figure 1 Schematic of annular reactor

limit (MDL) was 0.03 mg/L for the laboratory spectrophotometer and 0.09 mg/L for the portable apparatus.

DOX samples were collected in dark bottles, headspace-free, and analyzed in duplicate according to method 5320 B (*Standard Methods*, 1995) using an adsorption module[*] and analyzer.[†] The method accuracy and MDL were 1 and 3 µg/L Cl^-, respectively. Sodium arsenite ($NaAsO_2$) was used as a dechlorinating agent (Hureiki et al., 1994). Prior to analysis, samples were acidified to a pH of 2 by adding concentrated nitric acid and were processed within seven days.

To identify the parameters influencing the changes in chlorine and DOX concentrations in the distribution system, other analyses were

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[*] Dohrmann AD-3, Rosemont Analytical Inc., Dohrmann Division, Santa Clara, Calif.

[†] DX-20A, Rosemont Analytical Inc., Santa Clara, Calif.

conducted: ultraviolet (UV) absorbance at 254 nm, TOC and dissolved organic carbon (DOC) by persulfate–UV oxidation according to method 5310 C (*Standard Methods*, 1995), biodegradable organic carbon (BOC) (Servais et al., 1989; Servais et al., 1987), HPC by membrane filtration and incubation on R2A medium at 20°C for seven days according to method 9215 D (*Standard Methods*, 1995), and total direct counts (TDC) (Hobbie et al., 1977). Control blanks for all parameters were processed during each sampling. Ultrapure water* was used for DOX analyses. The reagents used for the sample collection and preservation were also used for the control blanks.

Statistical Analyses and Modeling

Statistical analyses and modeling were conducted.† The database included all analytical parameters as explanatory variables, as well as the network in which the data were collected (Chomedey or Pont-Viau), sampling dates, water temperature (cold, lukewarm, or warm), size of pipe (large or small diameter), summation of chlorine doses added in postdisinfection, disinfectant residuals measured in the distribution system, total RT including treated water clearwells and distribution system, and cumulative ratios of S to V and ST to V in which S is the cumulative surface of pipe, T is the RT in the distribution system, and V is the cumulative volume of water. The $ST{:}V$ ratio takes into account the contact between the pipe surface and the water and was calculated for each sampling point by determining the main route of the water to the sampling points. Cumulative ratios were obtained by adding all ratios from the different pipe sections.

RESULTS AND DISCUSSION

Table 1 shows the characteristics of the water leaving the treatment plants during the samplings. During data analysis, results were grouped in three categories to account for variation in temperature-related parameters and seasonal changes in organic matter: cold (0°–4°C), lukewarm (8°–14°C), and warm water (16°–24°C).

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* Milli-Q, Millipore Canada Ltd., Mississauga, Ontario

† SAS system, SAS Institute, Cary, N.C.

Table 1 Water characteristics at the treatment plant effluents

	Pont-Viau		Chomedey	
Parameter*	Mean	Range	Mean	Range
Chlorine dose (mg/L Cl_2)	1.78	1.10–3.23	2.26	1.60–3.05
Temperature (°C [°F])	7.4 (45.3)	0.7–23.1 (33.3–73.6)	9.9 (49.8)	0.5–19.7 (32.9–67.5)
pH	8.1	7.5–8.6	8.2	7.8–8.7
Turbidity (ntu)	0.11	0.08–0.13	0.10	0.04–0.15
UV absorbance (cm^{-1})	0.27	0.021–0.032	0.028	0.022–0.041
TOC (mg/L)	3.1	2.8–3.3	2.8	2.5–3.0
DOC (mg/L)	3.1	2.7–3.5	2.8	2.5–3.0
BOC (mg/L)	0.5	0.3–0.8	0.4	0.2–0.5
HPC (cfu/mL)	78	29–140	30	3–83
TDC (cells/mL)	1.9×10^5	7.11×10^4 -1.65×10^5	2.12×10^5	5.09×10^4 -4.76×10^5

*UV—ultraviolet, TOC—total organic carbon, DOC—dissolved organic carbon, BOC—biodegradable organic carbon, HPC—heterotrophic plate count, cfu—colony-forming units, TDC—total direct count

Chlorine

Figure 2 shows the free chlorine concentrations in the main pipes, small-diameter pipes (150 mm [6 in.]), dead ends, and batch incubations. RTs are represented on the abscissa with the origin corresponding to the entrance of the distribution system (treatment plant effluent). Concentrations of free chlorine varied from the MDL of 0.09 mg/L Cl_2 to 1.02 mg/L Cl_2. Total chlorine concentrations followed the same pattern as free chlorine, with combined chlorine levels (i.e., difference between the free and total chlorine concentrations) in the range of 0.05–0.17 mg/L Cl_2. Chlorine decay with increasing RT in the distribution system or distance from the treatment plants has also been observed by others (Chen & Weisel, 1998; Meyer et al., 1993; Capellier et al., 1992; Mathieu et al., 1992; Colbourne et al., 1991; Desjardins et al., 1991; Olivieri et al., 1986).

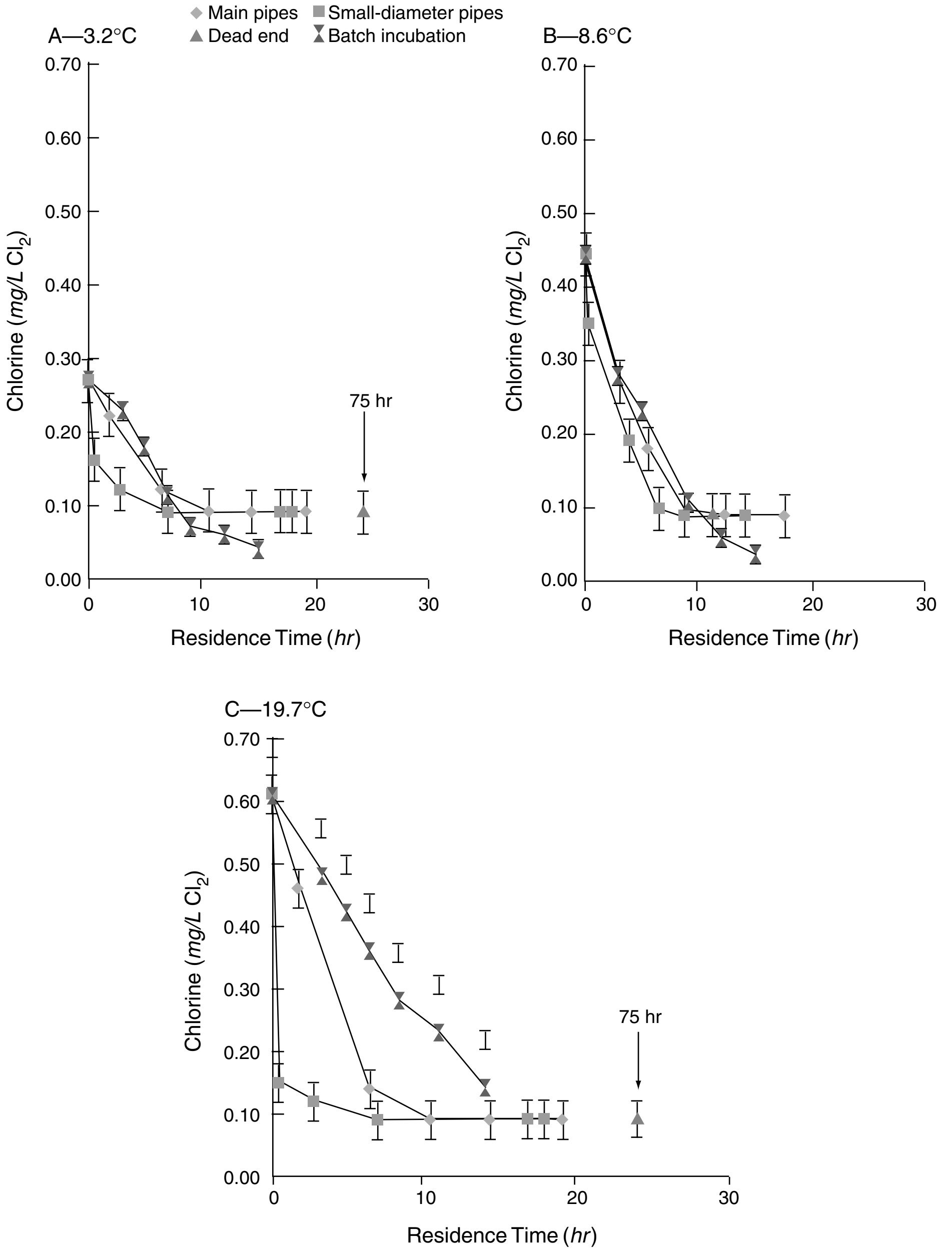


Figure 2 Free chlorine residual as a function of residence time in the distribution system

The chlorine decay with increasing incubation time in the batch incubations was attributed to the chlorine demand of the organic and inorganic chemicals (effect of the constituents found in the water). Pipe influence (i.e., biofilm, materials, corrosion by-products, pressure, and agitation) was indicated by the higher chlorine demand measured in the distribution system than in the batch incubations. The differences between the distribution system and batch incubation results were more significant in warm water. Chen and Weisel (1998) and Desjardins et al. (1991) observed similar trends.

Chlorine demand was higher in the small-diameter pipes than in the main pipes, particularly in warm water (Figure 2). The pipe effect may be attributed mainly to the chlorine demand of the biofilm (Rompré et al., 1995; Mathieu et al., 1992; LeChevallier et al., 1990; LeChevallier et al., 1988a; LeChevallier et al., 1988b) and corrosion by-products present on some materials (Rompré et al., 1995; LeChevallier et al., 1993; LeChevallier et al., 1990). Kiene (1994) demonstrated that hypochlorous acid may be transformed into chloride ion in the presence of ferrous iron that is oxidized into ferric iron. In the small-diameter pipes close to the treatment plant, important chlorine demand was measured, which can be attributed to the pipes' highly corrosive material (unlined gray iron). In the main pipes composed of ductile cast iron or steel-reinforced concrete, important chlorine demand did not occur because these materials are covered with a cement layer that prevents corrosion. Visual observation of pipe sections confirmed the presence of corrosion by-products (mainly iron compounds) in small-diameter pipes and their quasi-absence in main pipes.

Chlorine decay

In order to distinguish the influence of the water, biofilm, and corrosion by-products, the authors compared the rate of free chlorine decay in main pipes, small-diameter pipes, and batch incubations. Rates of chlorine decay were estimated by dividing the slope of the linear sections of the free chlorine curves by the total surface area of pipe and by the RT. For the main pipes, the chlorine consumption attributed to the pipe material was assumed to be negligible because the concrete pipes studied had been on line for several years, and corrosion was negligible compared with the small-diameter pipes. Therefore, the difference between the chlorine demands measured in the batch incubations (effect of the water only) and in the main pipes (effect of the water and biofilm) was used to estimate the chlorine demand of the biofilm alone. In the small-diameter pipes, subtracting the chlorine demand of the batch incubations from that measured in the distribution system provided an estimate of the chlorine demand associated with the material (unlined gray iron with corrosion by-products) and biofilm.

In the main pipes, chlorine decay rate was 0.29–1.00 $\times 10^{-5}$ mg L^{-1} m^{-2} hr^{-1} Cl_2 for water and biofilm and 0.13–0.19 $\times 10^{-5}$ mg $^{L-1}$ m^{-2} hr^{-1} Cl_2 for biofilm only. In small-diameter pipes, the chlorine decay rate was 120–190 $\times 10^{-5}$ mg L^{-1} m^{-2} hr^{-1} Cl_2 for water, material, and biofilm and 80–110 $\times 10^{-5}$ mg L^{-1} m^{-2} hr^{-1} Cl_2 for material and biofilm. These chlorine decay rates indicate that chlorine demand of the biofilm was much lower than that of the corrosion by-products. These calculations were performed with the data collected only from warm water samplings because the differences between the batch incubations, main pipes, and small-diameter pipes were more important at that time.

Statistical analyses and modeling of chlorine results

Because the free and total chlorine data did not follow a normal distribution, statistical analyses were performed on concentrations transformed by natural logarithms (ln Cl_2). Statistical analyses conducted on microbial parameters during the same study (Prévost et al., 1998) showed the need to stratify data by water temperature (warm, lukewarm, or cold), network (Chomedey or Pont-Viau), size of pipe (large or small diameter), and presence or absence of free chlorine residual (above or below the MDL). Thus, an initial analysis was performed with all the data obtained, followed by subsequent analyses after the data were subgrouped as a function of these factors.

The correlation matrixes, including Pearson's correlation coefficients (data not shown), indicated that RT and surface-to-volume ratios were the parameters that best explained the data, whether stratified or not. Bacterial counts (HPC and TDC) and organic matter levels (TOC and DOC) were also well correlated but mainly when the data were not stratified. These results can be explained by a co-correlation phenomenon—when chlorine concentration decreased with increasing RT, bacterial counts increased (Prévost et al., 1998). When only the data collected in the presence of free chlorine were considered, chlorine concentrations measured at the plant effluent were significant, probably because of the seasonal variability of the chlorine doses and consumption (i.e., higher doses were applied in summer to account for the higher chlorine demand measured in warm water).

Explanatory models including the coefficient of multiple correlation (R^2) were developed with all the data obtained, as well as with the data obtained in the presence of free chlorine only.

Eq 1 shows the model with all the data obtained:

$$\ln \mathrm{Cl_2} = -4.71 - 0.196\ \mathrm{TRT} + 2.68\ \mathrm{TOC} - 63.6\ \mathrm{UV} - 0.0287\ T° \ (R^2 = 0.756) \qquad (1)$$

in which TRT is the total RT (hr) including clearwells and distribution system, UV is the UV absorbance measured at 254 nm (cm–1), and $T°$ is the water temperature (°C).

Eq 2 shows the model with data stratified (presence of free chlorine):

$$\ln \mathrm{Cl_2} = -3.28 - 0.0101\ ST{:}V + 0.756\ \mathrm{TOC} + 0.325 \ln (\mathrm{Cl_2\ PE})\ (R^2 = 0.673) \qquad (2)$$

in which $ST{:}V$ is the ratio of cumulative pipe surface (S) × RT in the distribution system (T) to cumulative volume of water (V) and ln ($\mathrm{Cl_2}$ PE) is the natural logarithm of the free chlorine level measured at the plant effluent (mg/L $\mathrm{Cl_2}$). As for the correlation matrixes, modeling showed that in general, RT was the parameter controlling chlorine decay, explaining 64% of data variability. Organic matter concentrations were also correlated but explained only 7.7% of the data variability. When only the data collected in the presence of free chlorine were considered, the contact between the water and the pipes ($ST{:}V$) explained 42% of the chlorine decay.

DOX

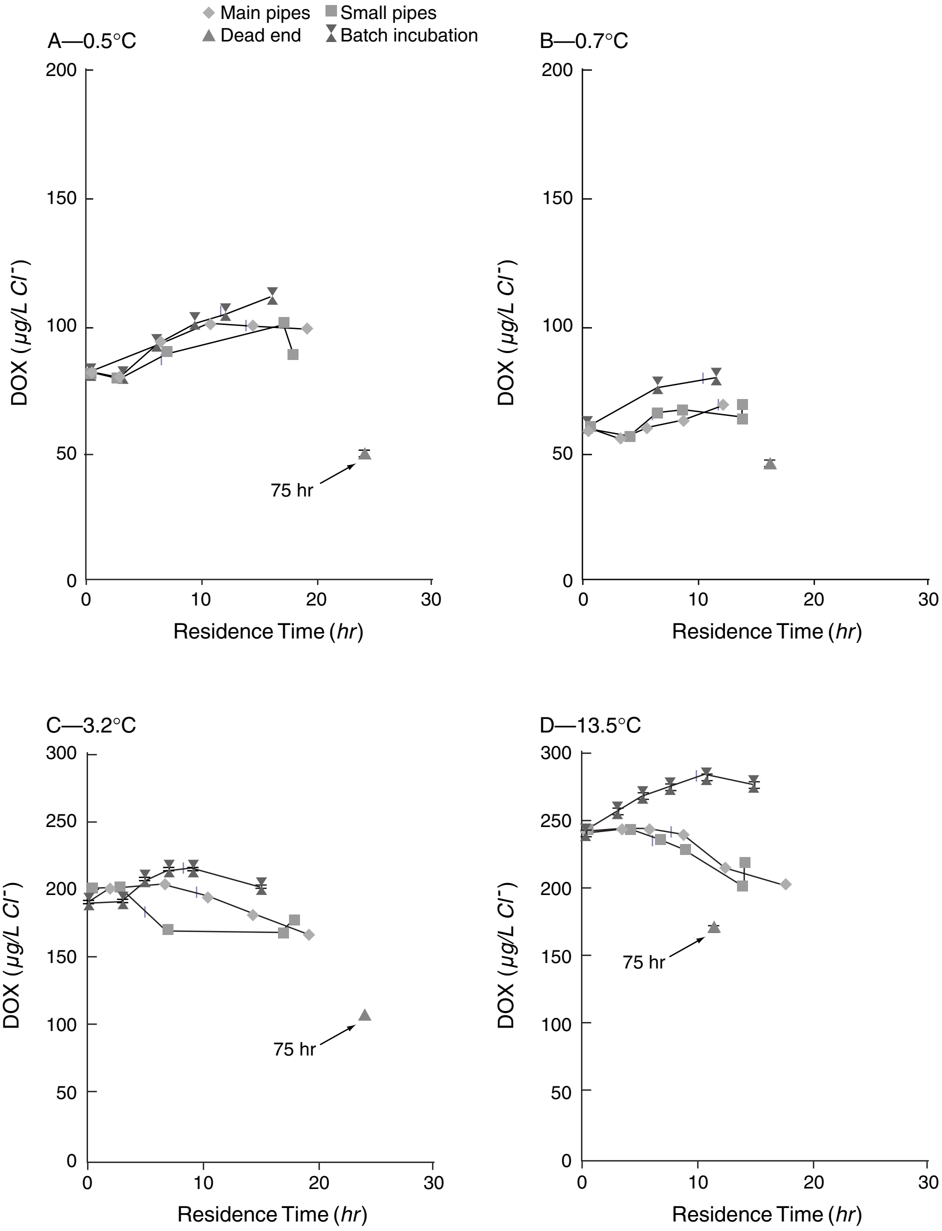
Although current and forthcoming regulations apply to specific contaminants such as THMs or HAAs, this chapter presents only DOX. Other chlorination by-products were examined during the study. Results showed that the molar ratio of the four THMs—chloroform, bromoform, and dichlorobromo- and chlorodibromomethane—to DOX (THM4:DOX) varied from 3.8% to 7.9%; the molar ratio of the four HAAs—monochloro-, dichloro-, trichloro-, and monobromo-acetic acids—to DOX (HAA4:DOX) varied from 1.7% to 5.2% (Baribeau, 1995).

Although the trends followed by specific DBPs can differ from those followed by DOX, DOX can still be considered a good surrogate for the summation of the THM4 or total HAAs, which constitute the regulated DBP parameters. In addition, Singer et al. (1995) observed good correlation between THM4 and DOX ($R = 0.868$), and HAA4 and DOX ($R = 0.911$) at treatment plant effluents and in distribution systems, with ratios of 15% for both THM4:DOX and HAA4:DOX. These ratios were much higher than those calculated by Baribeau (1995) partly because of the higher chlorine doses applied in the Singer et al. (1995) study (3.9 to 7.4 mg/L Cl_2).

Figure 3 shows some of the DOX data collected in main pipes, small-diameter pipes, dead ends, and batch incubations. On the left side of the vertical lines, a residual of free chlorine was measured, whereas on the right side, the chlorine concentration was below the MDL (0.09 mg/L Cl_2). In the presence of free chlorine, DOX concentrations generally increased in the distribution system in warm and cold water. In lukewarm water, levels remained stable. In the absence of measurable chlorine concentrations, DOX concentrations decreased with increasing RT in warm and lukewarm water; in cold water, the decrease in DOX concentrations was much less pronounced.

DOX concentrations were higher in the batch incubations than in the distribution system, which differs from the conclusions of Koch et al. (1991), who measured similar DBP concentrations (THMs, HAAs, haloacetonitriles, haloketones, chloropicrin, and chloral hydrate) in distribution systems and batch incubations (simulated distribution system tests). The differences can be explained in part by the higher chlorine doses applied in the Koch et al. study (3.3 to 4.0 mg/L Cl_2) and the higher disinfectant residuals targeted (> 0.2 mg/L Cl_2 after 96 hr). Meyer et al. (1993) found lower THM concentrations in batch incubations than in their pilot distribution system even though the chlorine doses applied were relatively similar to those added in the city of Laval treatment plants.

The differences in DOX concentrations measured in the distribution system and the batch incubations can be explained by the more rapid chlorine consumption in the distribution system, which led to lower DOX levels. In addition, results suggested that two phenomena occurred simultaneously in the distribution system: DBP formation in the presence of free chlorine and DBP decay. In the presence of chlorine and depending on the chlorine dose applied, the organic matter present, the water temperature, and other factors, an increase in DOX concentration can be observed when DBP formation exceeds decay (e.g., in the summer when the chlorine doses are high). When chlorine concentrations decrease to nonsignificant levels, the decrease in DOX already formed predominates. DOX decay may be attributed to several hypotheses, with the relative importance of each varying according to the conditions prevailing in the distribution system. These hypotheses (discussed in subsequent sections) include DOX chemical degradation, interaction



DOX—dissolved organic halogens

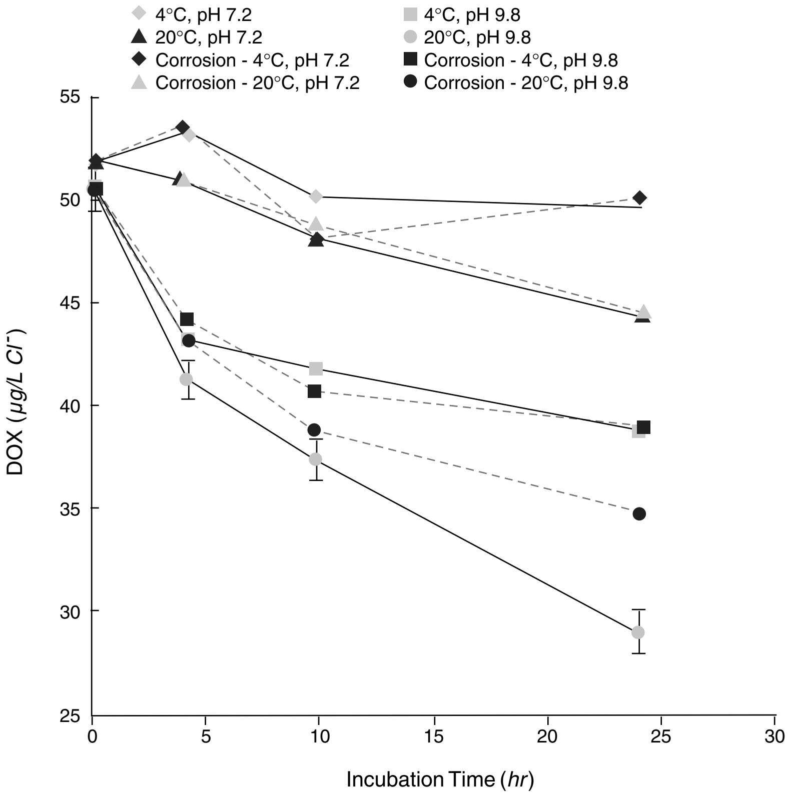
Figure 3 DOX concentration as a function of residence time with corrosion by-products, water pH, and adsorption in the biofilm matrix, biodegradation, and/or bioaccumulation by microorganisms.

Influence of corrosion by-products and pH on DOX

Laboratory experiments were conducted to evaluate the influence of pH and the assumption that corrosion by-products (mainly iron compounds) can interact with DOX. Water was collected at the Pont-Viau plant effluent, dechlorinated with $NaAsO_2$, and incubated for 24 hr at 4° and 20°C. For each of the temperatures, samples were incubated at pH of 7.2 and 9.8. This set of samples provided information about the change in preformed DOX in the absence of biofilm and corrosion by-products (i.e., effect of the water alone). Another set of samples included 1 g/L of corrosion by-products scraped from the inside of an unlined gray-iron pipe removed from the distribution system. The corrosion by-products were oven-dried at ~70°C to remove water and inactivate any microorganisms. This second set of samples indicated the change in preformed DOX in the presence of corrosion by-products and absence of biofilm, at 4° and 20°C, and at the minimum and maximum pH levels measured in the distribution system (7.2 and 9.8).

Figure 4 shows that corrosion by-products did not influence the DOX except when the incubation was carried out at an alkaline pH at 20°C. This unusual result can be explained by pH instability in the presence of corrosion by-products at high temperature and pH: during the 24-hr incubation, the pH dropped by more than 0.5 unit. Distribution system results confirmed the lack of influence of corrosion by-products on DOX, because similar DOX levels were measured in absence of residual in the small-diameter pipes made of unlined gray iron (a corrosive material) and in the main pipes presenting much less corrosion.

As Figure 4 indicates, DOX decay was much greater at high temperature and at pH 9.8 (24% at 4°C and 43% at 20°C) than at pH 7.2 (4.4% at 4°C and 15% at 20°C). In the absence of chlorine residual in the distribution system, however, DOX levels did not follow this trend. DOX concentrations usually decreased in the free-residual sections of the distribution system, whereas the pH decreased during some samplings, increased at others, or remained stable, indicating no specific trend (data not shown). In addition, pH variation was much higher during the laboratory experiment than in the distribution system (from 0.2 to 0.6 pH unit during each distribution system sampling, with minimum and maximum values of 7.3 and 9.0, respectively). It may be assumed, therefore, that pH by itself does not explain the change in DOX levels



DOX—dissolved organic halogens

Figure 4 DOX concentration as a function of temperature, pH, and the presence or absence of corrosion by-products

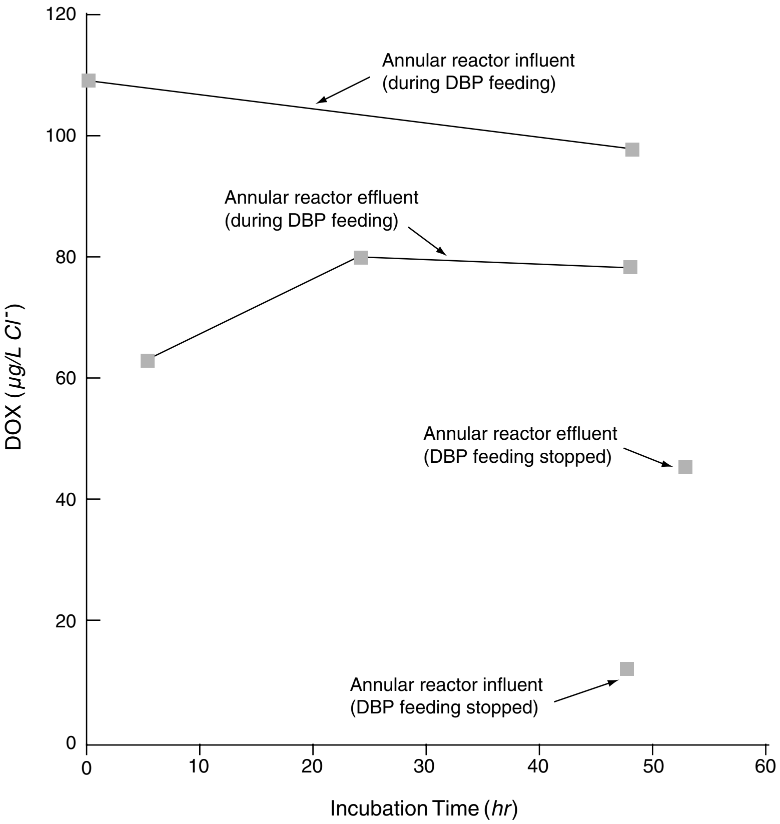
observed in the distribution system, a conclusion also reached by Meyer (1992).

Chemical degradation of DOX and influence of biomass

This hypothesis was addressed by conducting laboratory experiments in which a mixture of DBPs (chloroform, dichlorobromo- and chlorodibromomethane, di- and tri-chloroacetonitrile, 1,1-di- and 1,1,1-tri-chloropropanone, chloropicrin, di- and tri-chloroacetic acids) at a total concentration of 43 μg/L Cl^- was injected into a polycarbonate annular reactor (no corrosion) over a period of nearly 48 hr. A 48-hr period was selected in consideration of the hydraulic RT of the water inside the reactor (5 hr).

The 48-hr period allowed DBP seeding for several reactor detention times, which allowed significant contact between the DBPs and the biofilm. A longer seeding period was not recommended if results were to be compared with the RTs measured in the distribution system (< 20 hr).

A significant biofilm (0.31 μg/cm^2 C [2.0 μg/sq in. C]) developed inside the reactor after a colonization period of two months. DOX levels were measured at the annular reactor influent and effluent at regular intervals during the experiment, as well as 6 hr after DBP injection was stopped, to evaluate DBP release by the biofilm. As shown in Figure 5, DOX decay measured in the water feeding the annular reactor suggested chemical instability of some DBPs. Laboratory experiments conducted by Lévi et al. (1993) and Koch et al. (1988) demonstrated that in the absence



DBP—disinfection by-products, DOX—dissolved organic halogens

Figure 5 Influence of biofilm on DOX of chlorine, some chlorination by-products, including THMs, dichloroacetonitrile, bromochloroacetonitrile, and haloketones, were stable (< 20% of variability from the initial values) at 4°C and at pH of 7.5–7.7 for at least seven days. Other DBPs (e.g., 1,1,1-trichloropropanone, di- and trichloroacetonitrile, and chloropicrin) showed a gradual degradation. Chloropicrin decay was mainly associated with biodegradation rather than chemical instability (Lévi et al., 1993).

At the beginning of the experiment, DOX decay within the reactor was 42% but was only 20% after 48 hr of injection. The authors concluded that a biofilm never exposed to DBPs was better able to eliminate them (probably through biosorption and/or bioaccumulation) than was a biofilm that had been in contact with DBPs. It is possible that subsequent DBP decay was mainly attributed to biodegradation. The possibility that the DBPs are simply adsorbed in the biofilm matrix is supported by the significant release of DOX observed 6 hr after the DBP injection was stopped; DOX was released at a concentration four times higher (value of 45 µg/L Cl^-) than the background DOX concentration of the water normally injected in the annular reactor (12 µg/L Cl^-).

Statistical analyses and modeling of the DOX results

The correlation matrixes (data not shown) indicated that the seasonal effect (including the water temperature), the chlorine doses added in postdisinfection, and residuals measured at the treatment plant effluent were the parameters that exhibited the strongest influence on change in DOX concentration in the distribution system. These parameters were themselves correlated: as water temperature increased, chlorine doses added in postdisinfection also increased, which contributed to higher chlorine residuals. To a lesser extent, the organic matter content—TOC, DOC, and BOC—also influenced the data. When only the data obtained in the absence of measurable free chlorine concentrations were considered, bacterial counts (HPC and TDC) significantly explained the change in DOX concentrations in the distribution system, suggesting the influence of the microbial biomass on DBPs.

The authors developed several explanatory models (Table 2), which confirmed the observations drawn from the correlation matrixes. In general, water temperature was the parameter that best explained the change in DOX concentration, with more than 75% of the data variation as shown by the R^2 (also called percentage of explained variation). In warm water, the RT controlled DOX, whereas in lukewarm and cold water, the organic matter explained more than 84% of the data variation.

Table 2 Explanatory models for changes in DOX concentration

Condition	Model*	R^2
With all the data obtained	DOX = 107 + 9.77 $T°$	0.771
In the presence of free chlorine	DOX = –29.8 + 12.0 $T°$ + 4,460 UV	0.921
In the absence of free chlorine	DOX = 111 + 8.41 $T°$	0.757
In warm water	DOX = 358 – 4.15 TRT	0.967
In lukewarm water	DOX = –93.7 + 95.5 TOC + 2.98 $T°$	0.913
In cold water	DOX = –316 + 4,610 UV + 26.7 pH + 44.3 Cl_2 added	0.972

*DOX—dissolved organic halogen, $T°$—water temperature (°C); UV—ultraviolet absorbance measured at 254 nm (cm^{-1}); TRT—total residence time (hr); Cl_2 added—chlorine dose added in postdisinfection

These percentages resulted from intermediate calculations (not shown) as the statistical method used proceeded in a stepwise fashion.

SUMMARY AND CONCLUSIONS

This study showed changes in the concentrations of chlorine and DOX as a function of the water RT in two full-scale drinking water distribution systems. The effects of the constituents found in the water were distinguished from those of the distribution system, including pipes, diameter, material, biofilm, and corrosion by-products.

As expected, free and total chlorine concentrations decreased with increasing RT. The influence of pipes was important, with chlorine concentrations lowest in the small-diameter pipes, greater in the main pipes, and greatest in the batch incubations. In warm water, free chlorine demands attributable to the biofilm and corrosion by-products were evaluated at 0.16×10^{-5} and 96×10^{-5} mg L^{-1} m^{-2} hr^{-1} Cl_2, respectively. These rates of chlorine decay are rough estimates and do not take into account the following factors: biofilm density, which increased with RT even in the presence of free chlorine (Prévost et al., 1996); variation in water chemical characteristics; decrease in chlorine reaction kinetic with decreasing chlorine level; change in corrosion rate; and differences in hydraulic conditions between the batch incubations (completely mixed reactors) and the distribution system (plug-flow reactor) as well as water pressure and agitation occurring inside the pipes. Nevertheless, the important difference between the rate of chlorine decay associated with the biofilm and that associated with corrosion by-products indicates that pipe corrosion is the major factor influencing chlorine decay in the distribution system.

Pipe influence on DOX was shown by the differences observed between the batch incubation and distribution system samples. In contrast to the chlorine results, DOX concentrations were about the same in both small- and large-diameter pipes. In the distribution system, DOX concentrations generally increased in the presence of free chlorine and decreased when free chlorine concentrations were below the MDL. Results suggested that two phenomena occurred simultaneously in the distribution system—DBP formation in the presence of chlorine and DBP decay in the presence and absence of chlorine. DOX decay may be attributed to the chemical instability of some DBPs and the adsorption, biodegradation, and/or bioaccumulation of DOX in the microbial biomass.

Laboratory experiments and distribution system data indicated that in the case of the city of Laval distribution system, corrosion by-products and pH were not significant factors influencing DOX. Statistical analyses indicated that water temperature was the most significant parameter controlling DOX. Bench-scale experiments conducted to evaluate the influence of corrosion by-products and pH on DOX must be evaluated cautiously because these experiments were conducted in bottles (completely mixed reactors) and therefore did not consider the influences of pressure, agitation, and plug-flow hydraulic conditions of the distribution system. Use of an annular reactor to evaluate the influence of biomass on DOX allowed consideration of several hydraulic conditions encountered in full-scale distribution systems (including agitation and plug-flow conditions), but the influence of pressure was not taken into account.

Several tendencies observed during this study were reported as a function of water RT in the distribution system. Validation of these conclusions required that RTs be accurately calculated. The hydraulic models used to calculate RTs were state-of-the-art software programs designed for drinking water distribution system modeling, adapted to the city of Laval system's specific characteristics, and calibrated. Hydraulic simulations for RT calculations were performed with care and considered such factors as water routes in pipes, daily variations in water flow, pressure, and water consumption at each node and link.

Sampling locations were also carefully selected to obtain similar hydraulic conditions (e.g., water velocity in pipes, velocity profiles, and water consumption with time) and to avoid uncertain conditions such as flow inversion in pipes, low water velocity, extreme turbulence created by upstream hydraulic structures (e.g., valves, change in direction), blend of water with significantly different RTs, and areas close to the edge of pressure zones. In the case of sampling locations located in small-diameter pipe, water was first flowing in main pipes, then in small-diameter pipes. To ensure reliability between sampling locations, the small-diameter pipes selected were preceded by main pipes made of steel-reinforced concrete or smaller pipes made of unlined gray iron. Preferred locations were those with the longest route in small-diameter pipes.

In addition, treatment plant and distribution system operators and managers were asked to minimize any operational changes that might affect water quality and RT (e.g., water treatment, disinfectant dosage, status of valves in the distribution system). Utility personnel were also asked to inform investigators of any changes that occurred during the sampling period. Despite these precautions, tracer studies turned up differences between real and calculated RTs. However, complementary experiments demonstrated that water quality did not change significantly over time at each sampling location. The authors believe that any imprecision in RT estimation of some sampling locations was counterbalanced by the benefits of obtaining real data from full-scale distribution systems.

The differences observed between the batch incubation and distribution system samples for both the chlorine residual and DOX concentration raised questions about the validity of simulated distribution system tests. Numerous factors influence water quality in the distribution system—pipe materials, diameter, presence of corrosion by-products and biofilm, RT, water temperature, disinfectant residuals, and hydraulic conditions (e.g., completely mixed versus plug-flow reactors). As a result, each distribution system is complex and unique, and distribution system simulation in the laboratory is therefore subject to error. In addition, the DOX decay observed in the distribution system during this study has major implications with regard to the monitoring requirements of the Stage 1 Disinfectants/DBP Rule (USEPA, 1998) and upcoming Stage 2 Disinfectants/DBP Rule because sampling locations in the distribution system influenced the data obtained and, therefore, compliance with the regulation.

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CHAPTER · 10

Methodology for Selecting Distribution System Sites for DBP Monitoring

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BACKGROUND

With the promulgation of the Stage 1 Disinfectants–Disinfection By-products (D–DBP) Rule and the future promulgation of the Stage 2 D–DBP Rule, utilities are facing the challenge to select distribution system sites representative of worst-case conditions in terms of high trihalomethane (THM) and haloacetic acid (HAA) concentrations, high and low disinfectant residuals, and a range of hydraulic residence times. The USEPA recently developed a Guidance Manual to help the utilities select their distribution system sites for the Initial Distribution System Evaluation (IDSE).

Our team is currently working on an AwwaRF research project that studies factors affecting formation and decay of DBPs under actual distribution system conditions. While the goal of the USEPA Guidance Manual for IDSE is to help utilities select DBP monitoring sites in a full-scale distribution system, our study focuses on a relatively small area of the system to allow a comprehensive investigation of the factors affecting the formation and decay of several DBPs (THMs, HAAs, NDMA). Thus, the first step of our project consisted of selecting a general distribution system area for each utility. Once a distribution system area was identified, modeling was conducted to calculate the hydraulic residence time (water age) associated with each node in the distribution area. Monitoring sites were then selected based on water age and other hydraulic and water quality parameters. Five geographically spread utilities are participating in our project.

This chapter provides an overview of the regulations affecting distribution systems, and describes in detail the methodology used and the factors affecting the selection of distribution system sites used to monitor for THMs and HAAs under the Stage 2 D–DBP Rule.

INTRODUCTION

Disinfection by-products (DBPs) in drinking water have been a major subject of study over the past 30 years. The majority of those studies have focused on the formation and control of DBPs, primarily trihalomethanes (THMs) and more recently haloacetic acids (HAAs), in treatment plants. However, very little is known about the changes in DBP concentrations and speciation in full-scale distribution systems.

Because of the THM regulations (and the more recent HAA regulations), a significant database on DBP occurrence in tap water (i.e., distribution systems) has been developed, but most of this information relates to quarterly measurements made at selected locations in the distribution system. As an example, previous rules required the collection of grab samples at three monitoring locations with average residence times, and one site at a distant location. The recently developed Information Collection Rule (ICR) database contains DBP concentrations based on similar monitoring criteria. Accordingly, the database on THM and HAA occurrence is sparse from both a temporal (quarterly grab samples) and a spacial (only four locations per system) viewpoint.

Furthermore, preliminary evaluation of the ICR database indicates that the location of the maximum DBP concentration varies from one distribution system to another (close to the point of entry in some cases, or at "maximum" residence time in others). The location of maximum concentration also differs for both THMs and HAAs. This is the conceptual basis for the Initial Distribution System Evaluation (IDSE), which is being developed as part of the memorandum of agreement for Stage 2 of the Disinfectants–Disinfection By-products (D–DBP) Rule. The purpose of the IDSE is to revise monitoring locations used in the ICR to more accurately represent high concentrations of THMs and HAAs in distribution systems. In the Stage 2 D–DBP Rule, the THM4 and HAA5 compliance determination is based on a locational running annual average (LRAA) as opposed to the system-wide running annual average (RAA) used in the previous rules.

For the IDSE, systems can either (1) perform a one-year monitoring of their distribution system under the Standard Monitoring Program (SMP), which includes monitoring every two months over a one-year period of one to two points of entry, two average residence time sites, three high THM4 sites, and two high HAA5 sites, for systems serving more than 10,000 people; or (2) perform a System Specific Study (SSS), which includes the use of historical DBP data and water distribution system modeling.

If the root causes for the changes in DBP concentration and speciation are not better understood, however, utilities will be limited in their ability to determine the appropriate sampling points, as required by law.

Objectives

One of the goals of our AwwaRF project (No. 2770—Formation and Decay of DBPs in the Distribution System) is to determine a means of identifying the distribution system locations and times of maximum THM and HAA concentrations in order to address the Stage 2 D–DBP Rule requirements. For this project, points of entry, dead-end/remote areas, areas of low disinfectant residual, areas of high microbial counts, storage reservoirs, or chlorine booster stations were identified and monitored for concentrations and speciation of THM4, HAA9, and NDMA.

This chapter describes in detail the methodology used and the factors affecting the selection of distribution system sites to monitor for THMs and HAAs under the Stage 2 D–DBP Rule.

METHODOLOGY

Selection of Participating Water Utilities

It is understood that treatment processes affect subsequent water quality and chemical and biological reactions observed in the distribution system. However, in order to limit the number of factors investigated and focus this project on the distribution system, utilities were selected based on specific *finished* water characteristics, and not on the treatment processes used.

Four conditions were mandatory and common to all systems selected:

- Utilities must possess distribution system hydraulic models available for the determination of residence times (water ages) throughout the network.
- Systems must be simple and well characterized in order to maximize system control and limit the number of factors affecting the changes in water quality (i.e., limited number of pressure zones, limited number of interconnections).
- Water must not be blended in the distribution system (i.e., untreated groundwater mixed with treated surface water in the distribution system) to reduce system complexity and better separate and identify fundamental causes of DBP fate and transport in distribution systems.
- DBP levels must be moderate to high: THM4 > 40 μg/L and HAAs > 20 μg/L.

The following criteria or *intersystem variables* were also considered:

- Secondary disinfectant types, free chlorine or monochloramine.
- Moderate to high assimilable organic carbon (AOC) or total organic carbon (TOC) level.
- Presence of brominated DBPs in some sites.
- Variation in geographical locations to include different seasonal patterns and temperatures.
- Different pH levels, including different corrosion control techniques.
- Inclusion of a chloraminated site that converts to free chlorine periodically to avoid potential nitrification episodes.
- Inclusion of a system converting from free chlorine to chloramines, if possible.

Selection of Sampling Locations

Within each distribution system studied, the sampling locations were carefully selected to provide as complete an understanding as possible of the fate of DBPs considering the complexity of full-scale distribution systems. The sampling locations were based on the following *intrasystem variables*:

- Various water residence times (water ages) from close to the treatment plant to dead ends. The utilities' hydraulic models were used for this evaluation.
- Consideration of the pipe material and diameter. Samples are collected in both large-diameter pipes (main pipes) and small-diameter pipes.
- Preferably sampling locations within the same pressure zone.
- Areas with low or nonmeasurable disinfectant residuals (spatially and temporally).
- Critical areas of the network, such as locations with coliform or nitrification occurrences, or presence of specific DBPs that might not be present throughout the entire system.
- One system includes a booster chlorination station, a storage reservoir, and a combined booster chlorination station and storage reservoir. Samples are collected upstream and downstream of these facilities.
- Areas outside the zone of influence of potential major interference from external routine activities such as autoflushes.

System maps, historical distribution system water quality data, distribution system hydraulic models, and other pertinent information were used for this purpose.

Determination of water ages at the sampling locations

The following is a summary of the procedure used to determine the water ages in each distribution system and select the sampling locations.

UTILITY A—CHLORAMINE SYSTEM. Utility A uses H2ONET hydraulic model, and includes water age estimation. This hydraulic model was calibrated by comparing model predicted system pressures and tank hydraulic grade lines with those measured in the field.

Water ages were estimated at average, minimum, and maximum daily flows, under a typical yearly average flow rate condition. To calculate the water ages, H2ONET uses a numerical scheme called the Discrete Volume Element Method (DVEM). Within each hydraulic time step when flows are constant, DVEM computes a shorter water quality time step and divides each pipe into a number of completely mixed volume segments. Results include an estimate of the water age at each junction or node (at the head of each pipe segment) and an estimate of water flows leaving the node.

Thirteen sampling sites were selected for this study: these include the plant effluent, sites with various water ages ranging from 0 to >120 hours, sites with similar water age and varying pipe diameters, and sites known to experience some HAA degradation. All sites selected are located as far as possible from the zones of influence of the seven autoflushers that are located on the distribution system to avoid noncompliance episodes with the Total Coliform Rule.

UTILITY B—FREE CHLORINE SYSTEM. This utility uses the WaterCAD 4.5 hydraulic model to model the distribution system and estimate the water age. To obtain the range of water ages at various locations of the distribution system, the model was run for 14 days (336 hours), and the water ages were obtained at the minimum-, average-, and maximum-flow hours on the 13th day (i.e., hours 315, 327, and 320, respectively). The model was run for three different flow scenarios, corresponding to the minimum, average, and maximum daily flows (as reported during the previous calendar year). Water ages were then obtained for five scenarios, as follows:

- Minimum daily flow (minimum-flow hour)
- Average daily flow and:
  - Minimum-flow hour
  - Average-flow hour
  - Maximum-flow hour
- Maximum daily flow (maximum-flow hour)

The “maximum,” “average,” and “minimum” flow hours correspond to specific times of the day when the total system flow is expected (on average) to be at its maximum, average, and minimum for that day, respectively.

UTILITY C—FREE CHLORINE SYSTEM. The Hastead hydraulic model was used to model the distribution system and estimate the water age. Water age was obtained at average flow demand condition only. Ten distribution system sites were selected along a 30-in.-diameter ductile iron transmission, along with four other sites located in 4-in.-diameter PVC pipes. The water age along the main pipe ranges from 0 to 47 hours. All sites are located in one pressure zone fed by a single pipe coming out of the water treatment plant. The zone sampled does not have pumping stations or storage reservoirs.

UTILITY D—CHLORAMINE SYSTEM. A section of Utility D's distribution system was selected for this project. The section is simple, with no influence from storage reservoirs or booster stations. The section is fed most of the time by a single source. The hydraulic model (H2ONET) was run for 72 hours at the average flow demand. The water ages were calculated over the 72-hour period, and the evaluation was then focused on the minimum, average, and maximum hourly flow demands. Flow distributions and directions were also calculated in this part of the system. Water ages at the minimum, average, and maximum hourly flow demands were estimated for all nodes. Twelve nodes were selected and the distribution of the water age over the 72-hour simulation period was plotted (diurnal distribution). Eight of those sites were located in a transmission main, while the others were representative of smaller-diameter pipes.

UTILITY E—CHLORAMINE SYSTEM WITH TEMPORAL SWITCH TO FREE CHLORINE. The hydraulic model (EPANET) was run for 168 hours (one week) at the average demand measured in the summer of 2000. The range of water ages was determined at minimum-, maximum-, and average-day demand of the fourth and fifth days of this average week. A total of 14 distribution system sites were selected, representative of various water ages, pipe diameters, and problem areas.

Figures 1 to 4 present the distribution of water ages that are likely to be observed at four of the systems sampled.

SNAPSHOT OF RESULTS FROM FULL-SCALE DISTRIBUTION SYSTEM EVALUATIONS

Changes in Water Quality With Increasing Water Age

Water quality data are presented as a function of estimated average water age calculated at each sampling location. As much as possible, the data were separated by pipe size.

Trihalomethanes

As expected, THM4 concentrations increased with water age in the free chlorinated distribution systems (Figure 5). In the chloraminated systems,

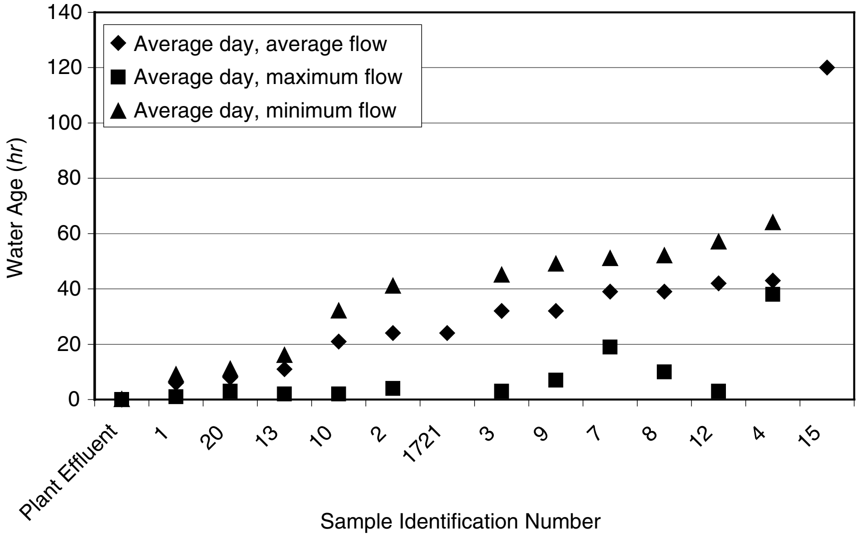

Figure 1 Distribution of water ages at Utility A's sampling locations

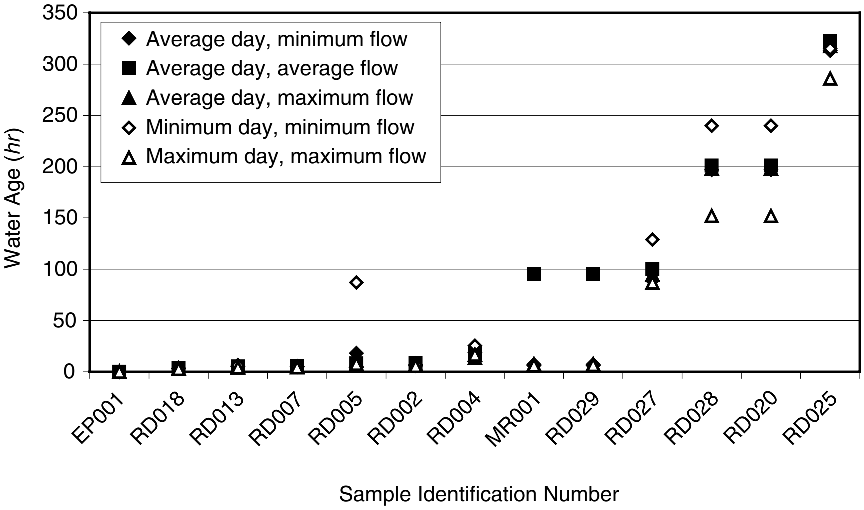

Figure 2 Distribution of water ages at Utility B's sampling locations

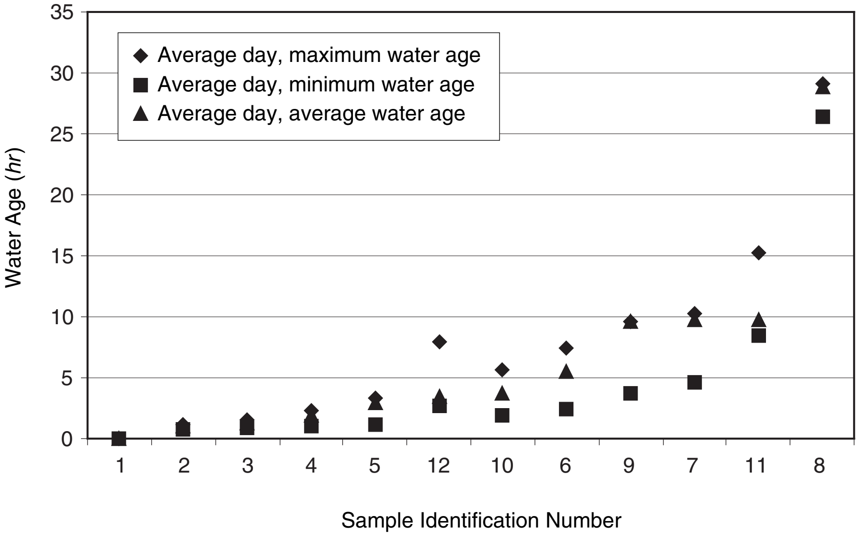

Figure 3 Distribution of water ages at Utility D's sampling locations

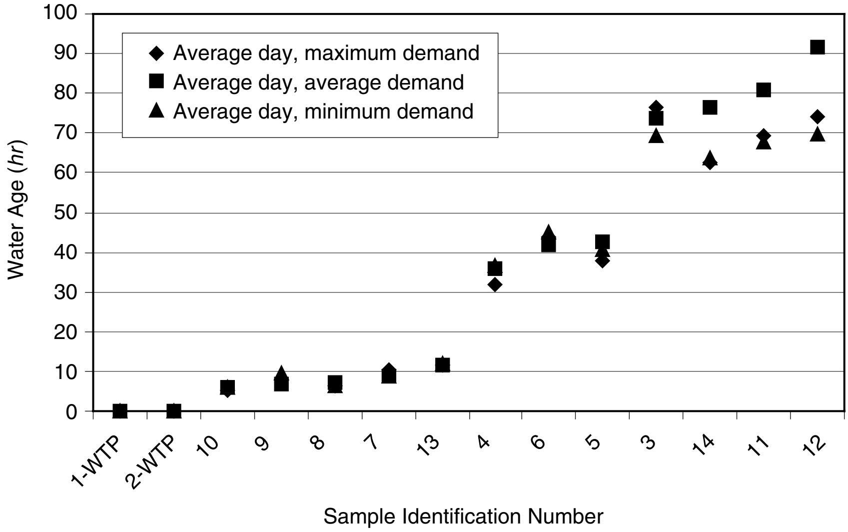

Figure 4 Distribution of water ages at Utility E's sampling locations

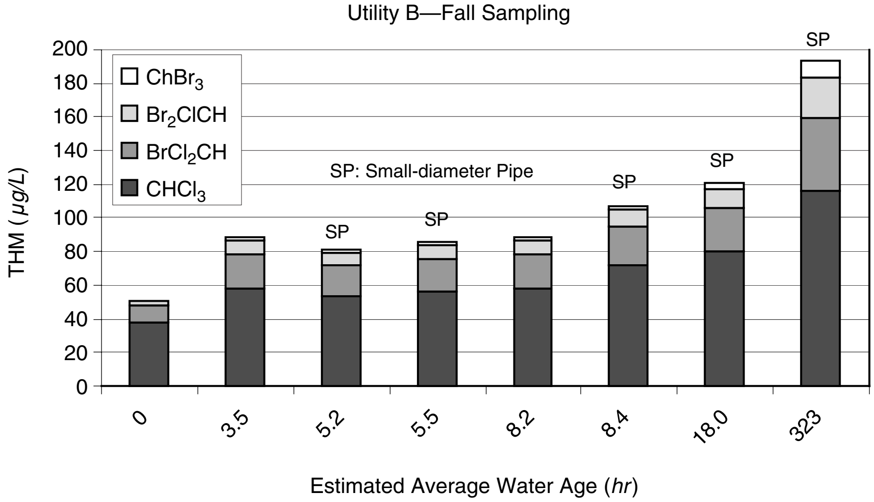

Figure 5 THM4 concentration as a function of water age in a free chlorinated system

the increase in THM4 concentration was much more subtle in the fall sampling and nonexistent in colder water (Figure 6).

Non-detects were reported according to the ICR. A minimum reporting level (MRL) of 1 μg/L was considered for each THM species, and any value below this MRL was reported as 0.

Haloacetic acids

In the free chlorinated systems, HAA concentrations showed different trends. They were relatively stable in the fall sampling (temperature of 13° to 19°C), with lower concentrations in the most remote distribution system locations. The decrease in concentration was mainly associated with lower X2AA levels. In colder water (temperature of 5° to 12°C), concentrations tend to increase slightly (Figure 7). Further data are required to confirm these trends. In the chloraminated systems, HAA concentrations are relatively stable in both the fall and winter seasons, with a slight decrease in concentration at the most remote distribution system locations (Figure 8).

Non-detects were reported according to the ICR recommendations. The MRLs considered for each HAA species are the following:

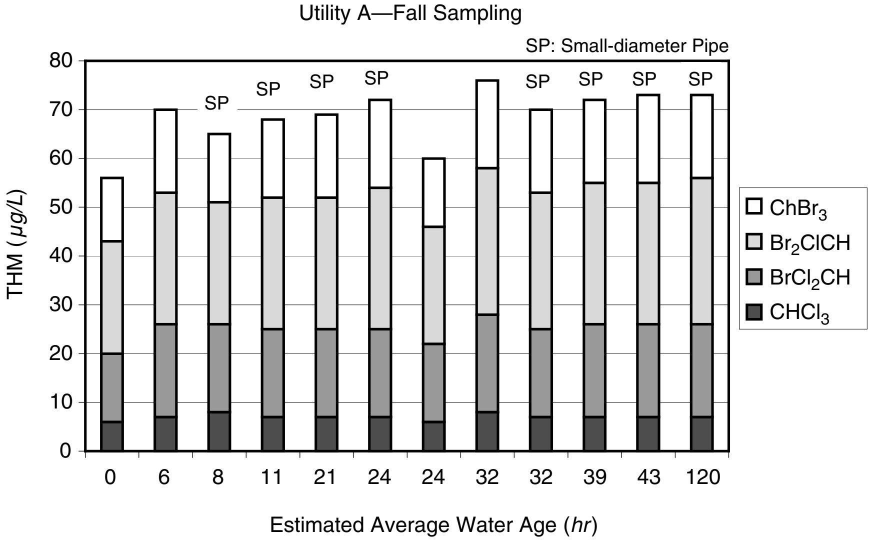

Figure 6 THM4 concentration as a function of water age in a chloraminated system

ClAA: 2 µg/L	Cl_2AA: 1 µg/L	Cl_3AA: 1 µg/L
BrAA: 1 µg/L	Br_2AA: 1 µg/L	$BrCl_2AA$: 1 µg/L
	BrClAA: 1 µg/L	Br_2ClAA: 2 µg/L
	Br_3AA: 4 µg/L	

Any value below the MRLs listed above was reported as 0.

Special Topics

Effect of booster chlorination stations and storage reservoirs

Two reservoirs of Utility B's distribution system are included in the monitoring program: a 2 MG, ground-level reservoir with booster pumps and without a chlorination station (referred to as "Reservoir 1"), and a 0.1 MG, elevated reservoir with a chlorination station and without booster pumps (referred to as "Reservoir 2"). The state of the reservoirs (emptying or filling) is being documented during every sampling. Figures 9 and 10 present representative THM4 and HAA9 results obtained.

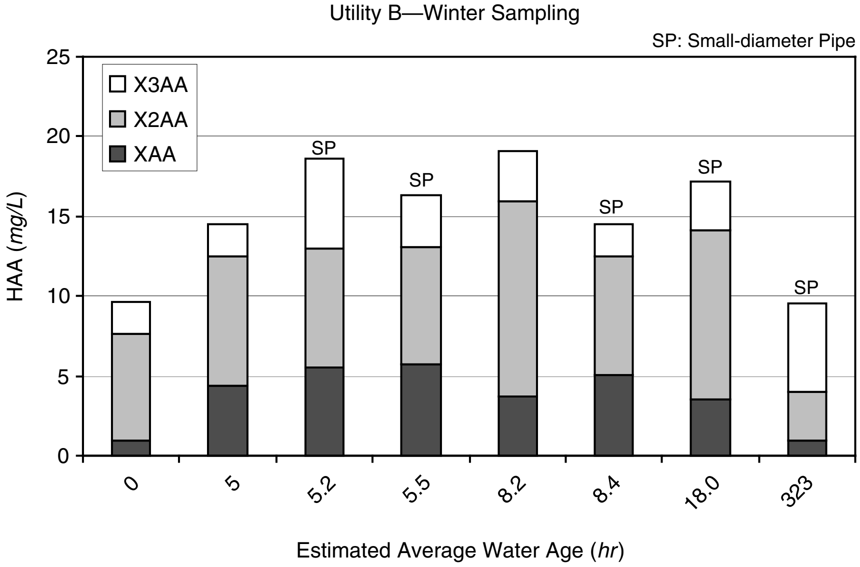

Figure 7 HAA9 concentration as a function of water age in a free chlorinated system

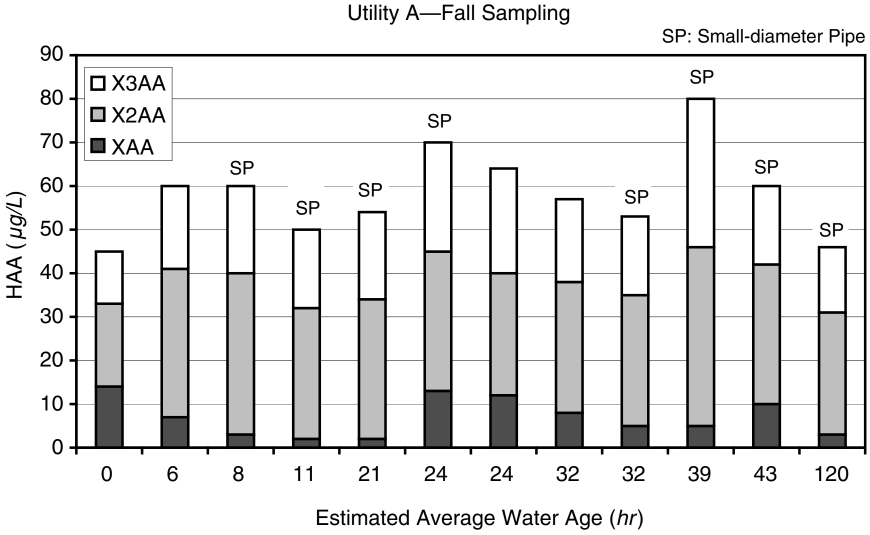

Figure 8 HAA9 concentration as a function of water age in a chloraminated system

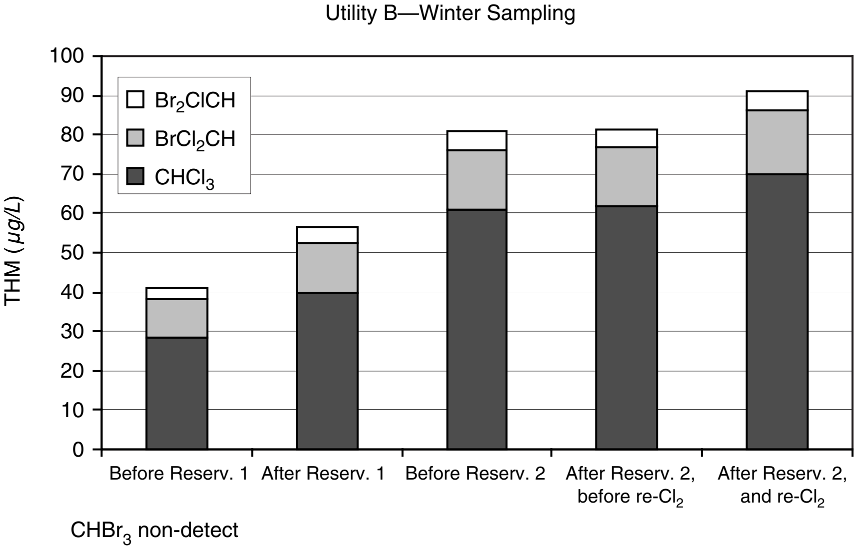

Figure 9 Effect of storage reservoirs and booster chlorination on THM4 concentrations

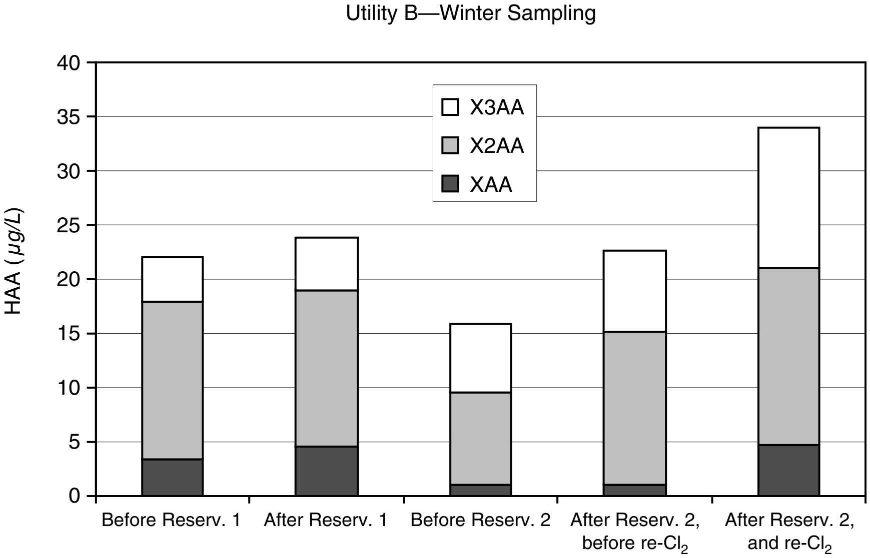

Figure 10 Effect of storage reservoirs and booster chlorination on HAA9 concentrations

CONCLUSIONS AND FUTURE WORK

This project aims at providing a clearer understanding of the formation and decay of DBPs in full-scale distribution systems to help water utilities select their DBP sampling locations in light of the Stage 2 DBP Rule. This chapter presented the methodology used to select the sampling locations in five full-scale distribution systems studied in this project. Some of the THM4 and HAA9 results obtained were presented. Results of a one-year monitoring program are being compiled and will be presented in subsequent presentations. Once all of the data are available, we will develop a simple and friendly operational tool to help utilities determine the locations of maximum THM and HAA concentrations in their distribution system.

The comments and views detailed herein may not necessarily reflect the views of the AWWA Research Foundation, its officers, directors, affiliates, or agents, or the views of the U.S. government.

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CHAPTER · 11

Seasonal Variations of Trihalomethanes and Haloacetic Acids Within Water Distribution Systems: A Case Study in Québec (Canada)

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BACKGROUND

Chlorination By-products in Drinking Water

Many types of techniques are used to disinfect water: chlorination, ozonation, ultraviolet radiation, and other disinfectants, including chloramines, chlorine dioxide, and potassium permanganate. Chlorine is the most commonly used disinfectant in conventional water treatment processes because of its low cost, its capacity to deactivate bacteria, and because it ensures residual concentrations in municipal distribution systems to prevent microbiological contamination. Research undertaken during the 1970s revealed the formation of disinfection by-products (DBPs) as a result of the reaction of chlorine with natural organic matter (NOM) in water (Rook, 1974). Figure 1 presents the principal chlorination by-products (CBPs) occurring in drinking water. According to more recent toxicological and epidemiological studies, some of the DBPs resulting from water chlorination, such as trihalomethanes (THMs) and haloacetic acids (HAAs), represent health risks to water consumers (Cantor et al., 1998; Bove et al., 1995). Some of these studies have found associations between high levels of DBPs and increased risk of cancer and adverse pregnancy outcomes.

In North America, the U.S. established the Disinfectants–Disinfection By-products (D–DBP) Rule, in two stages. The first stage of the D–DBP Rule establishes maximum contaminant levels (MCL) of 80 μg/L and 60 μg/L for THMs and HAAs, respectively, based on the running annual average of seasonal samples (USEPA, 1994). The second stage of the rule, to be published in the course of 2002, will propose a locational approach

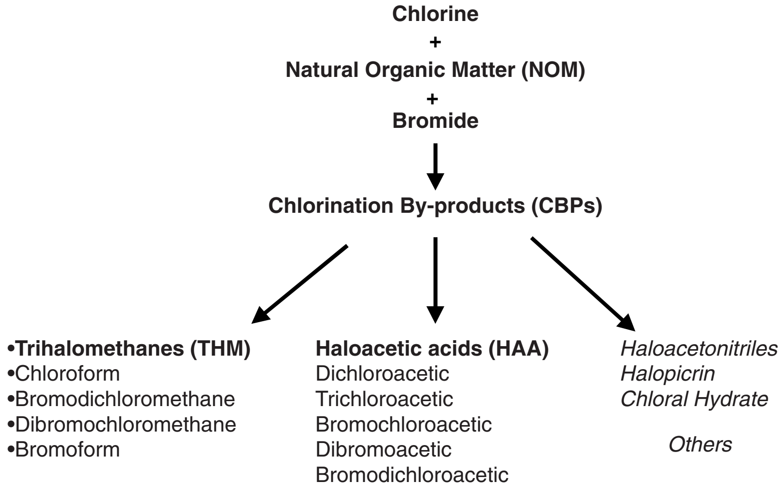

Figure 1 Main chlorination by-products in drinking water (adapted from Oxenford, 1996)

for CBP compliance within the distribution system (Sharfenaker, 2001). The government of Canada recently set out drinking water guidelines stating a maximum acceptable level of 100 µg/L for THMs (Health Canada, 1996). Maximum HAA levels are expected in a forthcoming update of the Canadian guidelines. In the province of Québec, in 2001, the Ministry of the Environment modified the annual concentration for THMs (based on four seasonal samples) from 350 µg/L to 80 µg/L (Gouvernement du Québec, 2001).

The trend toward updating CBP regulations constitutes a considerable challenge for utility managers who are faced with balancing microbial and chemical risks: ensuring a microbiologically safe water while minimizing the occurrence of CBPs in the distribution system. These will be demanding tasks, due to the complexity of the mechanisms responsible for residual chlorine decay and CBP occurrence in water systems. To reach their goals, water utility managers will require more detailed knowledge about the formation and evolution of CBPs in distribution systems.

FORMATION AND EVOLUTION OF CBPS IN DISTRIBUTION SYSTEMS

Past investigations have suggested that the occurrence of CBPs in chlorinated water may vary significantly according to season and geographical location in the distribution system (Rodriguez and Sérodes, 2001; Chen and Weisel, 1998; Arora et al. 1997; Singer et al. 1995). These temporal and spatial variations are due to changes in raw and treated water quality as well as in operational parameters related to chlorination. The measurable operational parameters that influence CBP occurrence in distribution systems are chlorine dose, water temperature, pH, and travel time of water within the system (also referred to as contact time or residence time). In terms of water quality, it has been established that fulvic and humic constituents of organic matter constitute important precursors for CBPs. Total organic carbon (TOC) and UV-254nm absorbance have been used as indicators of the presence of organic matter in drinking water. Bromide concentration in raw water can also affect CBP formation levels. Research based mainly on laboratory bench-scale data has shown that the higher values are for these parameters, the higher the concentrations of CBPs formed. In addition, bromide concentrations can significantly affect CBP speciation.

Once chlorine is applied, CBP concentrations may vary from the treatment plant to the extremities of the distribution system. Seasonal changes in water temperature may produce important changes in CBP levels. And CBP variations can be particularly important in water utilities where the residence time of water in the distribution system is considerable. Creating a strategy for meeting updated CBP regulations will depend on utilities having a better understanding of the impact that season and the location at which samples are collected can have on results.

FIELD STUDY IN THE QUÉBEC CITY REGION

The aim of this chapter is to paint a portrait of the spatial and seasonal evolution of THMs in two large utilities based on a high-frequency data collection program. The two utilities are located in the Québec City region (Canada). Because the collection of data related to CBPs in water utilities in the province of Québec was not mandatory before June 2001, there is currently very little information concerning variations in these compounds between the plant and the distribution system. Table 1 presents the general characteristics of the utilities under study.

The data for this analysis were generated in two different phases. Phase 1 represents a study of THM variations carried out between May 1999 and September 2000, whereas Phase 2 represents a simultaneous study of both THMs and HAAs carried out between September 2000 and September 2001. Phase 1 was designed to compare THM levels—on a seasonal basis—between the entrance point of the distribution system (following post-chlorination) and the system extremities. Phase 2 was designed as a study of the spatial evolution of THMs and HAAs based on sampling at several locations within the distribution system. The criteria for selecting sampling points within the distribution systems were numerous: (1) they had to be located at variable distances from the plant, to favor variable residence times of water; (2) they had to be located on-line from upstream to downstream; (3) at least one point had to represent the extremity of the distribution system; (4) all points had to be supplied directly by the treatment plant, so there was no influence of rechlorination facilities or water storage within the distribution system; and (5) all points had to be accessible for sampling the same day, every week over a year.

For both Phase 1 and Phase 2 from May to September—a period in which the variations of surface water temperature are greater—samples were collected weekly or twice per month in order to assess the variations in water quality in detail. From October to April 2000—the period when soil and surface water are covered by snow and ice—variations in water temperature are far less notable, thus samples were collected monthly. The collected samples were analyzed for different water quality and operational parameters: pH (pH-meter), temperature (portable thermometer), free residual chlorine (DPD titrimetric method, Standard Method 4500-Cl), total organic carbon—TOC (total carbon analyzer using unfiltered samples), UV-254 absorbance (UV/visible spectrophotometry), and THMs and HAA5 (E-capture chromatography, EPA 551, 2 and 552, 2 methods, respectively). The first three parameters were measured in the field, the others in the laboratory at Université Laval. The procedures used for the conservation of the samples between their collection and analysis were those recommended by *Standard Methods* (APHA, AWWA, WPCF, 1996) and are described elsewhere (Rodriguez and Sérodes, 2001).

Table 2 presents the average levels for indicators of CBP precursors in raw and treated waters. As observed, levels for TOC and UV appeared comparable in both utilities. However, applied chlorine doses in Sainte-Foy are appreciably lower than in Québec, which uses chlorine at the beginning and the end of the treatment.

Table 1 General characteristics of the two utilities under study

	Sainte-Foy	Québec
Water source	Saint-Lawrence River	Saint-Lawrence tributary
Treatment process	Pre-ozonation PCT treatment* Post-ozonation Post-chlorination	Pre-chlorination PCT treatment* Post-ozonation Post-chlorination
Flow rate	60,000 m^3/d	170,000 m^3/d

* Includes coagulation, flocculation, sedimentation, and filtration.

Table 2 Average values for indicators of organic matter and of chlorine dose for the two phases

	Raw Water			Treated Water*			Chlorine Dose (mg/L)†
	pH	COT *(mg/L)*	UV-254 *(cm^{-1})*	pH	COT *(mg/L)*	UV-254 *(cm^{-1})*	
Sainte-Foy	7.8.	3.63	0.175	7.8.	2.15	0.042	2.3
Québec	7.3.	4.31	0.202	6.6.	2.24	0.033	3.3

* Water following all physicochemical processes before post-chlorination.

† Includes pre-chlorination and post-chlorination.

Results for THMs in Phase 1

During the period under study, chloroform was the predominant THM compound in *both* utilities (about 80% of the THM content in Sainte-Foy and about 95% in Québec). The most important brominated THM was found to be bromodichloromethane, which, after chloroform, is also the second most important THM species from a toxicological point of view.

Given all sampling dates and points, levels for chloroform varied from 2 to 124 μg/L in Québec and from 4 to 97 μg/L in Sainte-Foy.

Important seasonal variations of THMs occurred in all the utilities during the period under study (Figure 2). As expected, levels of THMs were especially high during summer and fall and very low in winter. Indeed, the average levels of THMs measured in summer at the distribution system extremities were, depending on the utility, from 2.5

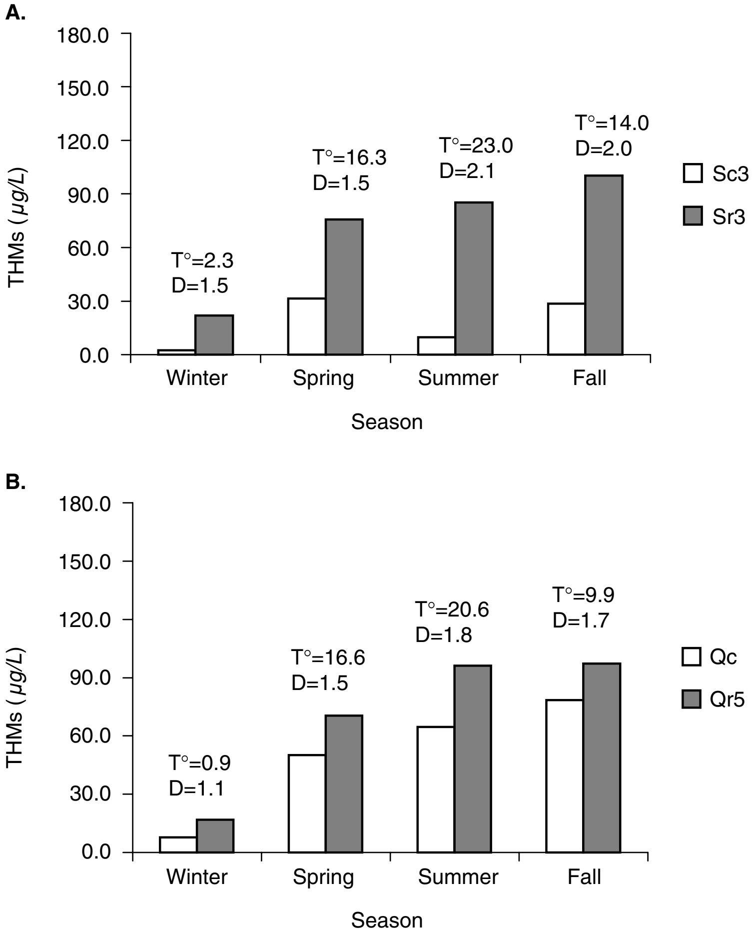

Figure 2 Seasonal variations of THMs: (A) Sainte-Foy (B) Québec (*Winter*: Dec., Jan., Feb.; *Spring*: Mar., Apr., May; *Summer*: Jun., Jul., Aug.; *Fall*: Sep., Oct., Nov.; Sc3 and Qc denote points at the entrance of the distribution system; Sr3 and Qr5 denote points at the distribution extremity) (T°: average temperature of treated water, °C; D: post-chlorination dose, mg/L)

to 5 times higher than the average levels measured in winter. These THM seasonal differences are much greater than those observed by other researchers working with THM data generated in water utilities in the United States and in Europe. The results can be explained by a number of factors. First, the average water temperature of treated waters was low during winter: for most sampling days, lower than 3°C and never exceeding 7°C. In addition, the transport of NOM from watersheds to surface waters is impeded by the ice cover that forms from December to March in southern Québec. Both factors—low water temperature and NOM content—result in lower chlorine demand in winter. Under such conditions, lower chlorine doses are applied during post-chlorination to maintain residual levels within the distribution system (even if, theoretically, higher doses are required in winter to deactivate microbial growth). The result is lower THM formation. Indeed, during the period under study, average chlorine doses were, according to the utility, from 1.3 to 3 times higher in summer than in winter.

However, as illustrated by Figure 2, average THM levels during fall and summer were comparable, even if average water temperatures in summer were significantly higher than in fall. This can be explained by the fact that during fall, the availability of NOM in watersheds increases because of vegetation decay. It was found that, depending on the utility, average values of TOC in fall were from 10% to 20% higher than in summer. Also, the post-chlorination dose applied in fall appeared comparable to doses applied in summer, and significantly higher than doses applied during spring, even if spring water temperatures were generally higher than fall water temperatures.

Results for THMs and HAAs in Phase 2

Figure 3 presents the spatial variations of THMs and HAAs in the distribution systems of both utilities during Phase 2 of the study. General levels of both CBPs, but in particular HAAs, were significantly lower in Sainte-Foy than in Québec. This finding is related to the levels of chlorine dose and also, undoubtedly, to the use of pre-ozonation to oxidize organic matter present in raw water. The indicators presented in Table 2, consequently, do not seem to be good surrogates for the reactive fraction of organic matter. In both utilities, an increase in THM throughout the distribution system can be observed, whereas HAAs seem to reach a maximum value and then decrease. As illustrated in Figure 4, the seasonal variations are also considerable, with values for Québec about 4 times higher in summer than in winter.

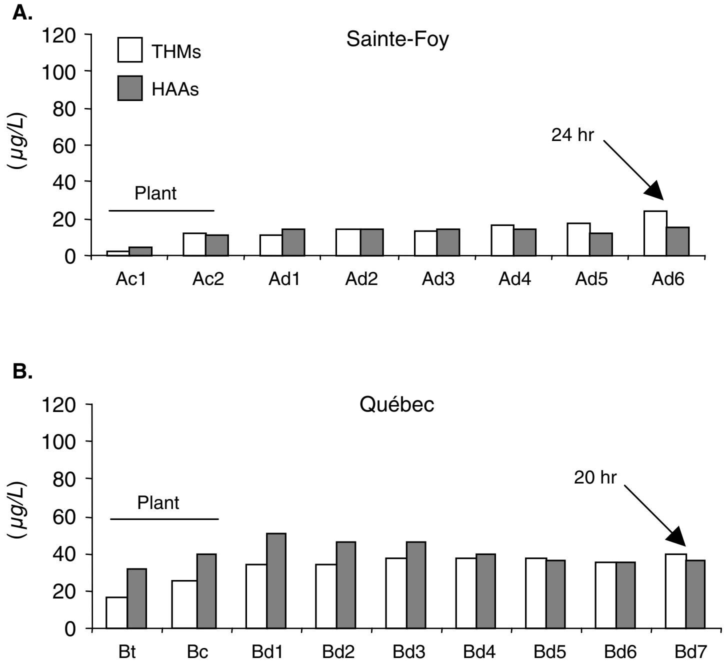

Figure 3 Spatial variations of THMs and HAAs in the distribution systems of both utilities (sampling points in X-axis are shown in increasing order of water residence time; approximate water residence time is shown for extremities)

Stronger correlations between HAAs and THMs appear at sampling points located near the plant as compared with points located in the extremities of the distribution system (Figure 5). It was also observed (data not shown) that correlations were also higher during summer months than in winter months. The variability in correlations underlines the importance of water residence time and seasonal conditions in the preponderance of both CBP species (THMs or AHAs).

The above results also suggest that in order to periodically measure THM levels in distribution systems—to comply with regulations—utility managers must select sampling points located at extremities (representing the higher residence time of water), where levels of THMs appear to be higher. However, the same location should not be used to monitor HAAs because there is an indication here that, as opposed to THMs, HAA concentrations—under certain specific operational conditions—may degrade within the distribution system (one hypothesis is that they are

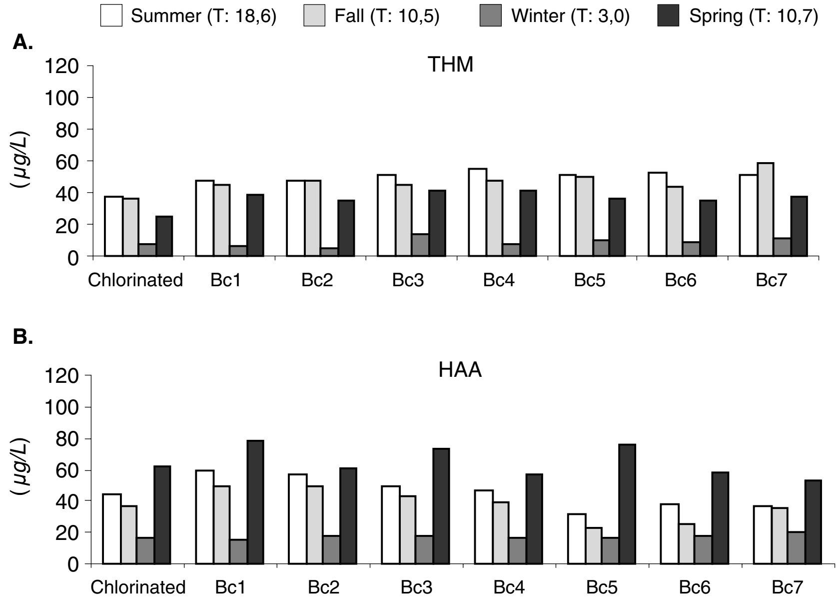

Figure 4 Spatial and seasonal variations of THMs and HAAs in Québec (*Winter*: Dec., Jan., Feb.; *Spring*: Mar., Apr., May; *Summer*: Jun., Jul., Aug.; *Fall*: Sep., Oct., Nov.; average water temperature, T°, is shown in legend)

biologically degraded). From a public health perspective, this would mean that strategies to reduce health risks associated with THMs in drinking water do not necessarily simultaneously reduce risks associated with HAAs. From the perspective of water quality monitoring in utilities, this would indicate that operators seeking to comply with eventual future regulations could not choose a single reference location within the distribution system for measuring these two by-products.

CONCLUSIONS

Results of Phases 1 and 2 of this study confirm those of other recent investigations, which indicate that CBP occurrence in urban distribution systems is highly variable. The two distribution systems in this study show CBP levels that are significantly higher in summer than in winter. THMs obviously increase between the plant and the distribution system extremities, until residual chlorine levels disappear. On the other hand, HAAs increase from the plant to the first sampling points, reach a

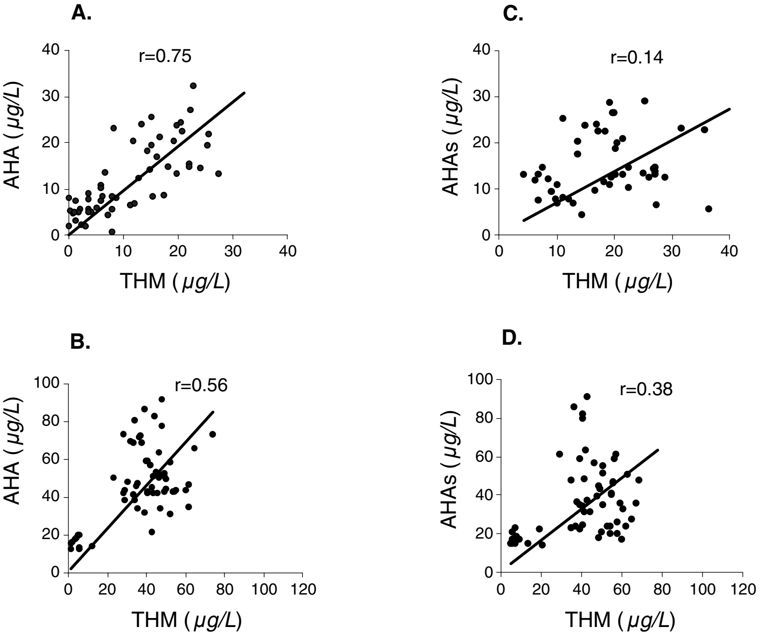

Figure 5 Correlations between HAAs and THMs according to location in the distribution system: (A) and (B) Sainte-Foy: entrance and extremity of the system, respectively; (C) and (D) Québec: entrance and extremity of the system, respectively

maximum, and then decrease, on approaching the extremity. This trend was particularly clear in Québec City, where global CBP levels were also notably higher.

Spatio-temporal variations of CBPs can have various implications. First, such variations could have an impact on how a utility complies with regulations. Generally, drinking water utilities' approach to CBP compliance is based on seasonal monitoring. Depending on the country, one or a few seasonal samples are required, according to the population served or the flow rate produced. Second, the consideration of such variations and the identification of the operational and water quality factors influencing them can be critical for drinking water utility managers trying to minimize CBPs in their distribution system. Third, variations of CBPs would directly affect human exposure to those substances in tap water, which would have important repercussions for exposure assessment in epidemiological studies.

Future research can now make use of the data generated in this study to model the occurrence of CBPs in distribution systems. The models must then be evaluated for their usefulness for water quality control and monitoring purposes, for regulation compliance, and for assessing CBP exposure in epidemiological studies. Including information about water residence time at different sampling points in the system (information gleaned through tracer field studies or hydraulic calibration) would improve the accuracy of the models that are developed.

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CHAPTER · 12

Disinfection By-product Analysis and Modeling in a Water Distribution System

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BACKGROUND

Minimizing the risks of disinfection by-products (DBPs) while maintaining adequate protection from microbial contamination is one of the dilemmas water utilities and regulating agencies are faced with today. The study of disinfection by-products (DBPs) has gained importance in the past decade because of the increased emphasis that regulatory agencies have placed on DBP control. DBPs are formed during water treatment when disinfectants and oxidants react with organic and inorganic matter. Since chlorine is used in a large majority of treatment plants in the United States, the chlorinated by-products are the most prevalent. The two chlorinated by-products that are regulated are trihalomethanes (THMs) and haloacetic acids (HAAs). The formation of both THMs and HAAs increases with time when a chlorine residual is present, leading to higher concentrations in the distribution system than in the finished water at the treatment plant. Regulations on DBPs are based on concentrations in the distribution system rather than at the end of the treatment process. Water quality models are often used in an attempt to analyze various water quality parameters in the distribution system. The goal of this research project was to develop a site-specific water quality model to simulate the formation of THMs and HAAs (all nine species) in the Gwinnett County, Georgia, water distribution system using a first-order saturation growth model with the distribution system modeling program WaterCAD®. These model predictions were compared with measured values from field sampling. The modeling effort produced results with varying success. The results from the HAA modeling appeared to be reasonably successful, having better agreement between predicted and measured concentrations than the THM modeling. While the measured and predicted results were statistically different, the results were of practical value in determining relative changes in water quality that can be expected at various locations throughout the distribution system. The results provided a reasonable indication of the effects of the distribution system on water quality and on the ability to model THMs and HAAs with distribution system hydraulic modeling software.

INTRODUCTION

Disinfectants such as chlorine and ozone are used in water treatment to combat the risks of microbial contamination, chlorine having been used in water treatment since 1908 (Betts, 1998). When disinfectants (or oxidants) are used, DBPs are formed. Several of these DBPs have been associated with various health effects, including cancer risks, cardiovascular disease, and adverse reproductive outcomes (Cohn et al., 1999). There are hundreds of DBPs that have been identified, the first of which was discovered in drinking water in the 1970s. Since that time, the water industry has faced the challenge of balancing its efforts to render drinking water free from microbial risks while reducing the formation of DBPs. Both THMs and HAAs are formed through the reactions of chlorine with natural organic matter (NOM). Since chlorine is the most widely used disinfectant in drinking water in the United States, THMs and HAAs are the most prevalent DBPs and were the major focus of this study.

THMs consist of four species: trichloromethane (chloroform, $CHCl_3$), bromodichloromethane ($CHCl_2Br$), chlorodibromomethane ($CHClBr_2$), and tribromomethane (bromoform, $CHBr_3$). The sum of these four species is called the total trihalomethane concentration (TTHM). There are nine HAAs: mono-, di-, and trichloroacetic acid (ClAA, Cl_2AA, and Cl_3AA); mono-, di-, and tribromoacetic acid (BrAA, Br_2AA, and Br_3AA); bromochloroacetic acid (ClBrAA); bromodichloroacetic acid (Cl_2BrAA); and chlorodibromoacetic acid ($ClBr_2AA$).

There are several factors that contribute to the formation and growth of DBPs in a water distribution system, including time, disinfectant dose, pH, temperature, precursor material, bromide concentration, and seasonal effects (Singer & Reckhow, 1999). Of these, time is one of the most important factors. The concentration of THMs and HAAs will continue to increase with time as long as there is a chlorine residual present. Growth of THMs with time tends to be very noticeable for the first several hours, and begins to taper off after two to three days (Kawamura, 2000). The formation of DBPs with time is what makes the distribution system an important factor in determining the concentration of DBPs that are found at the consumer's tap. The variation of hydraulic residence time leads to variations in THM and HAA concentrations throughout the distribution system.

Regulations

The enactment of the 1996 Safe Drinking Water Act (SDWA) Amendments outlined several changes to the existing water treatment regulations, including the Disinfectants–Disinfection By-products Rule (D–DBP Rule). The D–DBP Rule is broken into two stages (Stage 1 and Stage 2). The Stage 1 D–DBP Rule was promulgated on December 16, 1998 (USEPA, 1998). Compliance with the Stage 1 D–DBP Rule began in January 2002 (three years after promulgation) for all large subpart H systems. All small subpart H systems and all groundwater systems have until January 2004 (five years after promulgation) to comply (Pontius & Diamond, 1999). A subpart H system is a public water system that uses surface water or groundwater under the direct influence of surface water as a source; a large subpart H system serves 10,000 people or more, and a small subpart H system serves fewer than 10,000 people.

There are four DBPs that are regulated by the Stage 1 D–DBP Rule: THMs, HAAs, chlorite, and bromate. THMs are the only DBPs that were regulated prior to the Stage 1 D–DBP Rule. Of the nine possible HAAs, the sum of five of these (HAA5) are regulated (mono-, di-, and trichloroacetic acid, and mono- and dibromoacetic acid). The MCLGs and MCLs of these DBPs under Stage 1 are summarized in Table 1.

The MCLs for DBPs for the Stage 1 D–DBP Rule are based on a running annual average (RAA) of quarterly samples in the distribution system. The long-term proposed MCLs for the Stage 2 D–DBP Rule are expected to be the same as the Stage 1 D–DBP Rule, but compliance will be based on a locational running annual average (LRAA) instead of the RAA to reduce DBP peaks in the distribution system (Pontius, 2001).

Water Quality and Hydraulic Modeling

The concern of DBP concentrations and many other water quality parameters makes the use of water quality models an important part of treatment control. There are several different types of models available for many different water quality parameters. Several empirical and semi-

Table 1 MCLGs and MCLs for DBPs (Pontius & Diamond, 1999)

Disinfection By-products	MCLG mg/L	MCL mg/L
Total trihalomethanes	N/A	0.080
Chloroform	Zero	
Bromodichloromethane	Zero	
Dibromochloromethane	0.06	
Bromoform	Zero	
Haloacetic acids (HAA5)	N/A	0.060
Dichloroacetic acid	Zero	
Trichloroacetic acid	0.3	
Chlorite	0.8	1.0
Bromate	Zero	0.010

empirical models have been developed to predict the concentrations of THMs and HAAs throughout the treatment process and in the distribution system that attempt to incorporate the various parameters that affect DBP formation (organic content, disinfectant concentration, time, temperature, bromide concentration, and pH). These models are typically expressed as multiparameter power functions determined from regression analysis (linear and nonlinear).

First-order saturation growth models are often used to predict the growth of THMs and HAAs to a limiting concentration based on time and the initial concentration. These models are typically used with distribution network modeling programs to predict THMs and HAAs in the distribution system. These models typically incorporate the initial concentration, limiting concentration, rate coefficient, and time to describe the formation of DBPs. The rate of formation is described by $dC/dt = k(C_L\text{-}C)$ which, when integrated with respect to time (from $t = 0$ to $t = t$), yields:

$$C = C_L - (C_L - C_o)e^{-kt} \tag{1}$$

Where C is the DBP concentration at time t; C_L is the limiting concentration; C_o is the initial concentration; and k is the reaction rate coefficient. The units of these measurements can be specified, but concentration is typically given as µg/L and time is in either hours or days. The limiting concentration is generally taken as the final concentration of the kinetic study, i.e., the concentration after seven days in a seven-day kinetic study.

Distribution System Network Modeling

Distribution network models are used to analyze distribution systems for hydraulic and water quality performance. Historically these programs had only been used for hydraulic analysis, but they have recently become more popular in water quality analysis, since regulations are now often written based on concentrations found in the distribution system rather than at the end of the treatment process. Water quality modeling in a distribution system network predicts the variation of dissolved substances with time throughout the network under a set of hydraulic conditions and input patterns (Rossman & Boulos, 1996). A calibrated model can be used to predict the behavior of the distribution system under alternative conditions. Hydraulically, the network analysis is used to ensure proper design of new pipes, tanks, or pumps to be added to the distribution system. Network analysis is also used to test various scenarios, such as how the system handles fire flow conditions. The water quality portion of the network can be analyzed to determine how the system can be expected to perform in comparison with regulations or to determine the effects of changes in the treatment processes.

The computer software WaterCAD®, developed by Haestad Methods (Waterbury, Conn.), is an interactive hydraulic and water quality modeling program for water distribution systems. WaterCAD® is able to perform both steady-state analysis and extended-period simulations of distribution systems. It is able to track the growth or decay of dissolved substances and determine water age at any point in the network in addition to complete hydraulic modeling capabilities. Output is available in detailed reports, contour maps, and graphs showing the changes of any parameter with time.

GWINNETT COUNTY—LANIER FILTER PLANT

The 150 mgd Lanier Filter Plant, located in Buford, Georgia, currently supplies all of Gwinnett County's drinking water in addition to supplying some areas served by adjacent governments, serving a total of approximately 165,000 customers. The plant was originally built in 1975 and has gone through several upgrades and expansions to keep pace with the county's growing population. Lake Sidney Lanier supplies the raw water to the plant. It is a high-quality water source, low in suspended materials, bacteria, dissolved organics, and metals. Because of the low amount of suspended materials in the raw water, the Lanier Filter Plant uses direct filtration (coagulation, flocculation, and filtration).

Ozone is used at the Lanier Filter Plant for primary disinfection, with a typical dose of 1.0 mg/L and a minimum contact time of four minutes. Ferric chloride and cationic polymers are used for coagulation. Ferric chloride is added at a typical dose of 0.5 mg/L in the first stage of rapid mixing, and the cationic polymers are added at a typical dose of 1.5 mg/L in the second stage. The Lanier Filter Plant has 12 deep bed filters. The filter medium is 48 inches of anthracite coal and 12 inches of sand. The filters are operated at a maximum rate of 7.4 gpm/ft^2. During post-treatment, chlorine is added at a typical dose of 2.5 mg/L to maintain a residual throughout the distribution system. In addition to chlorine, fluoride is added (typically 0.7 mg/L), caustic soda is added (typically 7.0 mg/L) to adjust the pH, and blended phosphate is added (typically 2.5 mg/L) for corrosion control. The high-service pumping station delivers the water from the clearwells to the distribution system, which consists of over 1,900 miles of water mains, ranging in size from 78-inch transmission mains to 2-inch house lines.

MATERIALS AND METHODS

Sampling, Storage, and Analysis

Field sampling was conducted periodically, with a total of five sample sets taken, two summer days (July 3 and July 19, 2000), one fall day (October 31, 2000), and two winter days (January 3 and January 22, 2001). These sample sets provide a wide variation of temperature and seasonal conditions. Temperature, pH, conductivity, and residual chlorine were all analyzed in the field, while samples for THMs and HAAs were preserved and analyzed in the lab. Residual chlorine was determined by the DPD colorimetric method using a Hach Pocket Colorimeter™.

Each sample set included samples from the Lanier Filter Plant for both raw water and finished water, to show initial water quality, in addition to samples collected at 11 sampling points in the distribution system. The 11 distribution sampling points were the same ones used for the weekly HPC sampling conducted by the county. The location of each sampling point in the county distribution system is displayed in Figure 1.

THM samples were collected in 40 mL vials headspace-free and sealed with a PTFE/silicone septum. These samples were preserved with at least 100 mg of sodium thiosulfate per liter of sample to quench the residual chlorine and stored at 4°C. HAA samples were collected in 250 mL amber glass vials headspace-free and sealed with a PTFE/silicone septum. These samples were preserved with at least 100 mg of ammonium chloride per liter of sample to quench the residual chlorine and stored at 4°C. All sample lines were allowed to run for 10 minutes before sampling in order to flush stagnant lines.

The analysis of THMs was based on the Liquid–Liquid Extraction Gas Chromatographic Method detailed in *Standard Methods* 6232B (APHA, AWWA & WEF, 1995), with pentane used as the solvent. The analysis of HAAs was based on the "Improved Microextraction Procedure for the Analysis of HAAs (Yoo et al., 1997), which was presented at the 1997 AWWA Water Quality and Technology Conference. This method uses a liquid–liquid extraction with MTBE and derivatization of the acids with acidic methanol, similar to EPA Method 552.2. The method was designed to improve upon the reliability of the results achieved using acidic

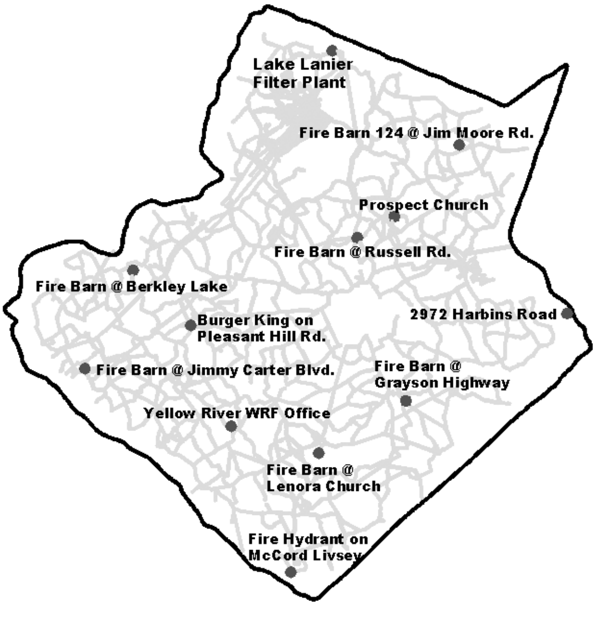

Figure 1 Location of distribution sampling points throughout Gwinnett County

methanol by increasing the derivatization efficiency and minimizing the evaporation of the MTBE extract.

Kinetic Study

A batch kinetic study was performed in order to determine how THMs and HAAs change with time. As discussed previously, the concentration of DBPs generally increases rapidly at first and tails off after a few days, and for this reason a first-order saturation growth curve is typically used to describe the growth of DBPs. This approach was used for the purposes of this study.

Finished water samples from the Lanier Filter Plant were taken and stored at 4°C and 25°C to determine the rate kinetics over a broad range of temperatures. In addition to these samples, an instantaneous finished water sample was taken for both THMs and HAAs when the samples were taken for the kinetic study, in order to determine the initial concentration. Samples for both THMs and HAAs were taken periodically throughout the 7-day test by quenching the chlorine residual and preserving samples at 4°C. These samples were taken at approximately 12 hr, 1 day, 36 hr, 2 days, 60 hr, 3 days, 4 days, 5 days, 6 days, and 7 days. Chlorine residual was measured each time a sample was taken in order to observe the decay of chlorine with time and ensure that a chlorine residual was maintained throughout the seven days. Two kinetic studies at both temperatures were performed in order to ensure that the results were consistent.

Once the concentration for each time step was determined, the first-order saturation growth curve was linearized, yielding

$$-\ln[(C_L - C)/(C_L - C_o)] = kt \tag{2}$$

The data were then fitted by linear regression to estimate the rate constant, k. The results of the kinetic study were used to determine the rate kinetics of each sampling day based on the average water temperature and the THM or HAA concentration of the finished water for that sampling day. These equations were entered into the water distribution system model in order to run the simulations for each sampling day.

Gwinnett County Water Distribution System Model

The Gwinnett County water distribution system network that was modeled included 2 reservoirs, 12 pump stations (including 2 high-service pump stations and 47 individual pumps), 12 tanks, approximately 1,850 pipes, and approximately 1,290 junctions. The distribution system model did not include the entire distribution network since it was unavailable at the time this study was completed. The network model included pipes of 12 inches and larger and some of the 8-inch and 6-inch pipes. The location of each sampling point can be seen in Figure 1. A different scenario was defined for each compound and each sampling day. There were a total of eight scenarios, five for THMs and three for HAAs. In WaterCAD® each scenario is defined by the various alternatives. There are physical, demand, initial settings, operational, age, constituent, trace, fire flow, cost analysis, and user data alternatives to be defined.

A steady-state calibration had been done previously on the Gwinnett County water distribution model by Severn Trent Pipeline Services, and this scenario was the source of several of the alternatives used in the water quality analysis for this project. An extended-period calibration would provide more confidence in the accuracy of water quality results; however, it was unavailable at the time the modeling was performed. The steady-state calibration tweaks the model to fit a "snapshot" view whereas an extended period calibration would tweak the model to fit time-varying data. The scenarios for each analysis were copied from the calibration scenario with adjustments made to the demand alternative, the initial conditions alternative, and the operational alternative. The physical alternative remained the same and the constituent alternative had to be developed.

In the constituent alternative, the rate kinetics for each sampling day for THMs and HAAs were specified for the bulk reaction. The limiting concentration was estimated based on the initial concentration multiplied by the ratio of C_L/C_o determined from the kinetic study. The extended-period simulation was run for three days in order to dampen the effects of the initial concentrations. The demand alternative was adjusted based on the flow from the day of sampling and the flow in the existing demand alternative. The flow patterns remained the same.

The initial levels of each tank and the initial status of each pump were set based on operator reports. The pump controls in the operational alternative were adjusted for each sampling day from the pump reports. The pump controls were time-based, turning on and off according to the actual pumping reports for each day (and the two preceding days) sampling was done. Once each scenario was defined, they were calculated for a 70-hr extended-period simulation. The concentration at the time step nearest to the sampling time was used as the estimate of the sampling concentration for each sample point to be compared with measured values. Every attempt was made to match the model to the actual conditions on the days of sampling.

RESULTS AND DISCUSSION

Field Measurements

A total of five sample sets were taken; each parameter was analyzed for all sampling dates except for HAAs. HAAs were measured for the last three sample sets. Tables 2 and 3 present an overview of the range of water quality parameters observed in the raw and finished water, respectively, throughout the sampling period. The SUVA was typically less than

Table 2 Observed raw water parameters

Water Quality Parameter	Range	Average
TOC (ppm)	1.4–2.2	2.0
UV-254 (cm^{-1})	0.022–0.030	0.027
SUVA (L/(mg C*m))	1.01–2.07	1.41
pH	7.02–7.34	7.15
Conductivity (μS/cm)	37.0–49.1	45.8
Water temperature (°C)	8.3–24.5	17.3

Table 3 Observed finished water parameters

Water Quality Parameter	Range	Average
TOC (ppm)	1.2–2.3	1.9
UV-254 (cm^{-1})	0.013–0.019	0.016
SUVA (L/(mg C*m))	0.62–1.58	0.93
pH	7.26–7.50	7.34
Conductivity (μS/cm)	48.5–65.5	59.4
Water temperature (°C)	5.2–25.9	16.5
TTHM (ppb)	3.8–7.5	17.0
HAA9 (ppb)	5.4–15.2	8.9
HAA5 (ppb)	3.9–11.5	6.6
Residual chlorine (mg/L Cl_2)	1.45–1.8	1.59
Volume of treated water (MG)	73.37–109.90	89.31

2 L/(mg C*m), indicating a low humic acid fraction of organic matter. This generally indicates a low removal of organic carbon (Letterman et al., 1999), which was apparent from comparing values of raw water TOC to those of the finished water. There was a large range of finished water THMs. The range of HAA values was much smaller than the range of THM values, which may be attributed to two factors, temperature dependence is not as significant for HAAs, and HAAs were not measured for the two summer samples, leading to a smaller range of temperatures and organic matter.

Many of the same water quality parameters measured for the raw and finished water were measured throughout the distribution system in order to observe the variations that can occur once the treated water leaves the plant. Table 4 provides a summary of the range of values measured throughout the distribution system on the various sampling days.

Table 4 Water quality parameters observed in the distribution system

Water Quality Parameter	7/03/00	7/19/00	10/31/00	1/03/01	1/22/01
Finished TTHM (ppb)	37.5	27.4	6.1	3.8	10.5
TTHM range (ppb)	45.1–113.3	36.0–102.0	14.0–75.9	5.0–16.5	12.7–35.5
TTHM average (ppb)	62.6	55.0	27.9	10.1	19.0
Finished HAA9 (ppb)	NA*	NA*	6.2	5.4	14.1
HAA9 range (ppb)	NA*	NA*	10.5–18.9	6.2–12.1	14.0–27.6
HAA9 average (ppb)	NA*	NA*	12.6	9.0	21.7
Finished HAA5 (ppb)	NA*	NA*	4.5	3.9	11.5
HAA5 range (ppb)	NA*	NA*	8.0–15.1	5.0–8.7	11.3–21.8
HAA5 average (ppb)	NA*	NA*	9.7	6.3	15.4
Residual chlorine range (mg/L Cl_2)	0.2–1.7	0.10–1.19	0.22–1.32	1.11–1.51	1.09–1.55
Average water temperature (°C)	23.6	25.1	21.9	8.1	8.7
pH range	7.44–9.64	7.59–9.44	7.39–9.27	7.33–7.77	7.57–9.33
Conductivity range (μS/cm)	60.6–89.3	62.4–80.9	65.6–78.7	63.7–69.2	64.1–76.6

*HAA samples were not taken for the July 3, 2000, and the July 19, 2000, sampling dates.

There was a wide range of TTHM values during the summer sampling and a smaller range of values during the winter sampling. The TTHM averages were much higher during the summer months than in the winter months. Figure 2 is a plot of the change in TTHM concentration versus the change in residual chlorine. The change in TTHM concentration and residual chlorine is the absolute value of the difference between the concentration at the sampling point and the concentration of the finished water for that sampling day. The general trend demonstrates that the greater the decay of chlorine, the greater the increase in THMs, which has been demonstrated before (Singer & Reckhow, 1999).

Both HAA5 and HAA9 are reported in Table 4 because HAA5 is the regulated value of HAAs while HAA9 is the sum of all the HAA compounds and may be a more appropriate indication of health risks. There was a large range of HAA concentrations throughout the distribution system (the maximum concentrations observed were almost twice as much as the minimum concentrations observed in each sample set), although this range was not as large as the range of THMs. Figure 3 presents a plot of the change in HAA9 versus the change in residual chlorine. It can be seen that there was a general trend showing an increase in HAAs with the decay of chlorine residual; however, the trend was not

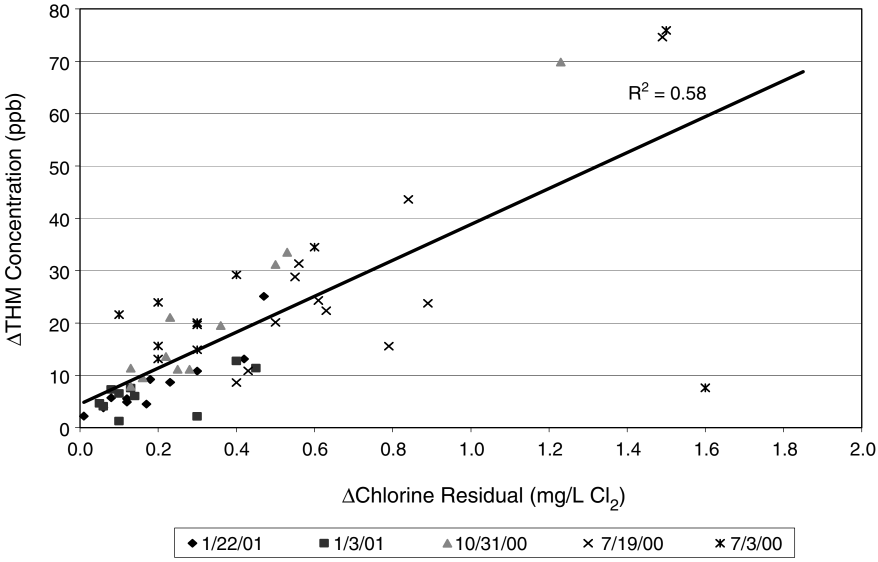

Figure 2 Change in TTHM concentration versus change in residual chlorine in the distribution system

as strong as that demonstrated with THMs and would also not be very useful for predicting HAA concentrations.

THM and HAA Speciation

Water utilities using source waters with low bromide concentrations find that the speciation of THMs and HAAs favors the chlorinated by-products. The speciation of the compounds can be an important factor to analyze since the various THM and HAA compounds have different health effects. Throughout the sampling conducted for Gwinnett County, chloroform was the most prevalent, comprising almost 75% of the TTHM concentration. Bromoform was not detected in almost all of the samples taken.

The speciation of the HAAs was similar to that of the THMs. Dichloro- and trichloroacetic acid combined for almost 75% of the HAA compounds (43% dichloro and 30% trichloro). Following these, bromochloro and bromodichloro were the next two most prevalent species. There was also a very small percentage of dibromoacetic acid measured. Monochloro-, monobromo-, chlorodibromo-, and tribromoacetic acid were not detected during the study.

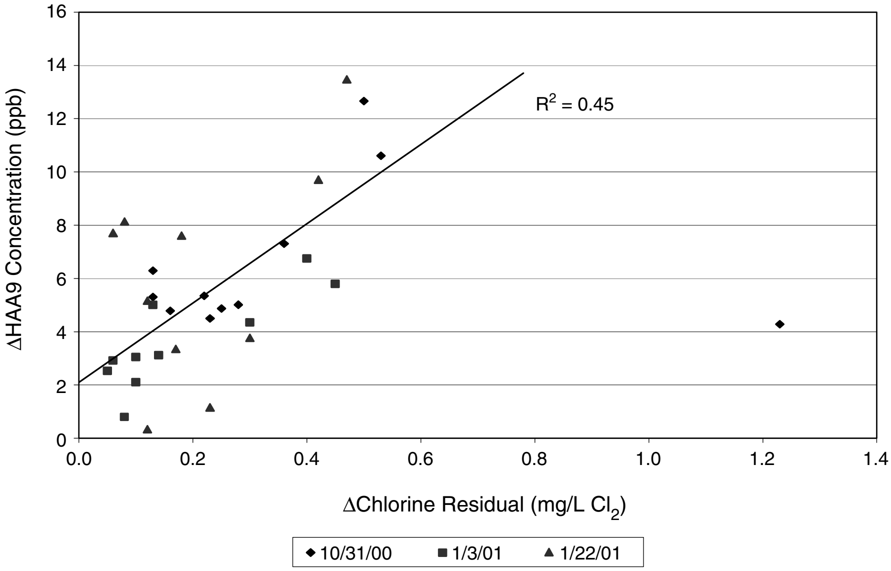

Figure 3 Change in HAA9 concentration versus change in residual chlorine in the distribution system

Batch Kinetic Study Results

Two batch kinetic studies were conducted in order to observe the change of THMs and HAAs with time. The results were used to estimate the rate constants for THM and HAA growth. The ratio of the limiting concentration to the initial concentration (C_L/C_o) was also estimated in order to determine the rate kinetics for each sampling day. Values that were determined to be temperature dependent were interpolated between 4°C and 25°C based on the average water temperature observed on each sampling day.

Figure 4 demonstrates the increase of the TTHM concentration with time at 4°C and 25°C. It can be seen that the TTHM concentration increased more rapidly and to a much higher final concentration during the 25°C tests. The solid lines on Figure 4 are the predicted values of TTHM concentration based on the initial concentrations, the water temperature, and the kinetic equation $C = C_L - (C_L - C_o)e^{-kt}$. In order to predict the change in TTHM concentration with time based on the initial concentration, two factors had to be determined. The first factor was the average ratio of the limiting concentration over the initial concentration (C_L/C_o), where C_L is estimated to be the maximum concentration found during the 7-day test. The concentration after 7 days was typically the highest measured concentration; however, due to experimental error and the flattening of the curve, there were instances when C_L was estimated to be the concentration after 5 or 6 days. C_o was estimated to be the concentration of the finished water. The average ratio of C_L/C_o was 6.5 at 25°C and 2.2 at 4°C. The values of C_L/C_o for each of the sampling days were interpolated between these values based on the average water temperature in the distribution system for each sampling day. The second factor necessary was the rate constant, k. This is determined by linearizing the data based on C_L and C_o. Because the linearization was based on the maximum concentration found and there was a large difference between the maximum concentrations at 25°C and 4°C, the k values were found to be relatively constant from 25°C to 4°C. For this reason, a single k value was determined based on a linear regression of all the linearized data. This makes only one factor, C_L/C_o, variable with temperature. Although the k values were estimated to be approximately equal, the kinetic rate still varied with temperature due to the effect of the limiting concentration (C_L) on the kinetics. The k value determined for the TTHM concentration was 0.43 day^{-1}.

Figure 5 demonstrates the change in HAA9 concentration with time at 4°C and 25°C. It was decided to model the formation of HAA9 because of the relative importance of all species. This is also a more conservative

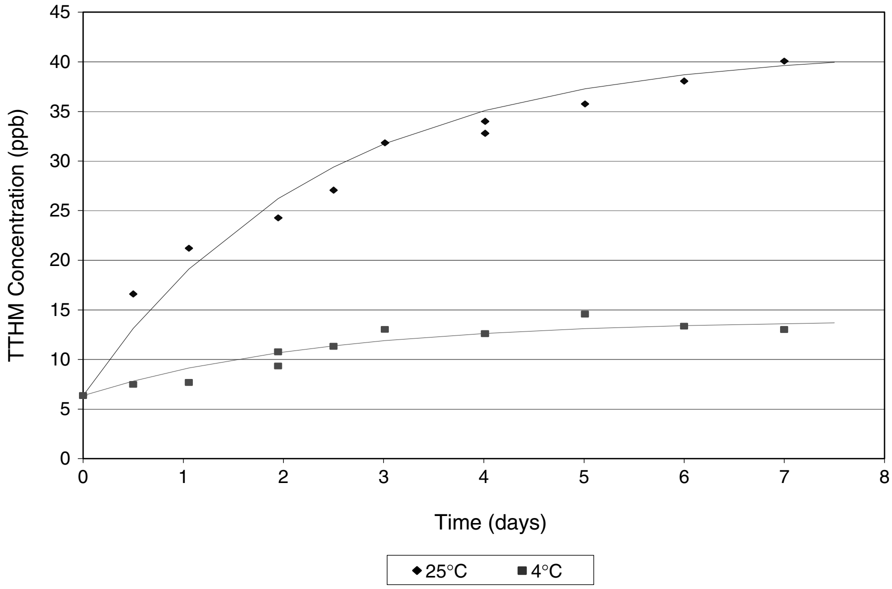

Figure 4 TTHM concentration versus time at 25°C and 4°C

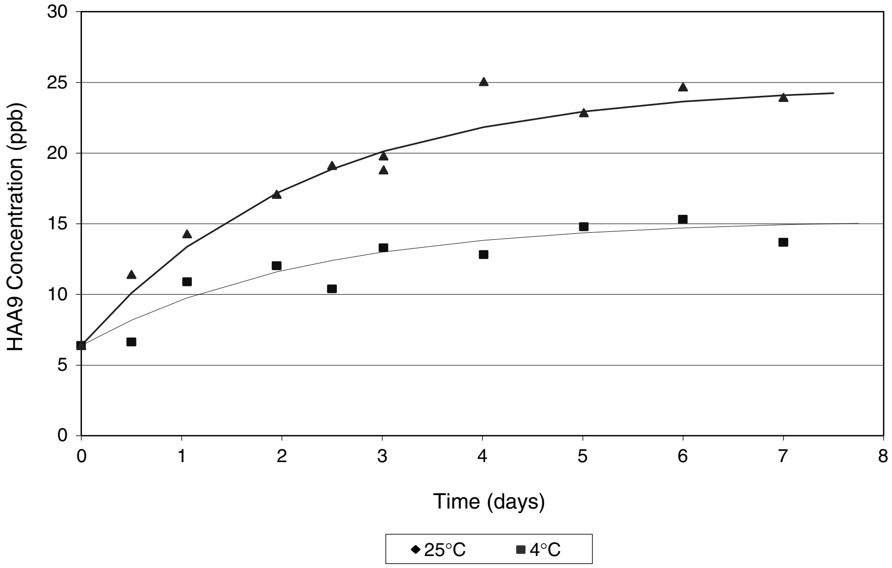

Figure 5 HAA9 concentration versus time at 25°C and 4°C (Kinetic Study 2)

approach, since if the HAA9 concentration is less than the MCL, then the HAA5 concentration is as well.

The relationship with time was similar to the TTHM relationship. As with the TTHM concentration, the HAA9 concentration increases more rapidly and to a higher final concentration during the 25°C test than in the 4°C test. The difference, however, was not as great as the difference observed from the THM kinetic study (the ratio of C_L/C_o was just over 1.5 times larger at 25°C than at 4°C for HAAs, and almost 3 times larger for THMs). This is expected since it has been demonstrated that HAAs are not as temperature dependent as THMs (Singer & Reckhow, 1999). The solid lines in Figure 5 are the predicted values of HAA9 concentration based on the initial concentration, the water temperature, and the kinetic equation $C = C_L - (C_L - C_o)e^{-kt}$. Since the same rate equation that was used for TTHM concentration was used for HAA9 concentration, the ratio of C_L/C_o and the rate constant k are the two factors that had to be determined. The ratio of C_L/C_o was 3.8 at 25°C and 2.4 at 4°C. The values of C_L/C_o for each of the sampling days were interpolated between these values based on the average water temperature. The slope of the linearized data was found to be the same at 4°C and 25°C as it was for the TTHM data. This was also because the limiting concentration was dependent on temperature. The k value was determined from a linear regression of the data from both the 4°C and 25°C tests. The k value determined for the HAA9 concentration was 0.45 day^{-1}.

The results from the kinetic studies were used to estimate the reaction rates for the various sampling days. The k values determined from the kinetic study were used, and the limiting concentration (C_L) was estimated from the interpolated value of C_L/C_o for each sampling date multiplied by the finished water concentration. Table 5 presents the estimated reaction rate expressions for each sampling date for THMs and HAAs, where C is the concentration of the DBP (THM or HAA). These are the reaction rates that were used in the distribution system simulations. The ratio of C_L/C_o was important for the characterization of the kinetics since the limiting concentration determines the reaction rate.

Distribution System Modeling of THMs and HAAs

The effectiveness of the modeling at predicting the measured concentrations of THMs and HAAs was promising, but not consistent. Figure 6 presents the plot of the predicted THMs versus the actual THMs for all the sampling points. The solid, straight line on the plot represents a 1:1 relationship. There are data points scattered on each side of the 1:1 line; however, almost all of the higher concentrations of THMs (from the

Table 5 Estimated THM and HAA reaction rates for the various sampling dates

Sample Date	Average Water Temp. (°C)	Interpolated C_L/C_o	Estimated C_L (ppb)	Reaction Rate
THMs				
July 3, 2000	23.6	6.2	232.5	dC/dt = 0.43*(232.5 – C)
July 19, 2000	25.1	6.5	178.1	dC/dt = 0.43*(178.1 – C)
October 31, 2000	21.9	5.9	36.0	dC/dt = 0.43*(36.0 – C)
January 3, 2001	8.1	3.1	11.8	dC/dt = 0.43*(11.8 – C)
January 22, 2001	8.7	3.2	33.6	dC/dt = 0.43*(33.6 – C)
HAAs				
October 31, 2000	21.9	3.7	22.9	dC/dt = 0.45*(22.7 – C)
January 3, 2001	8.1	2.7	14.6	dC/dt = 0.45*(14.5 – C)
January 22, 2001	8.7	2.7	38.1	dC/dt = 0.45*(38.6 – C)

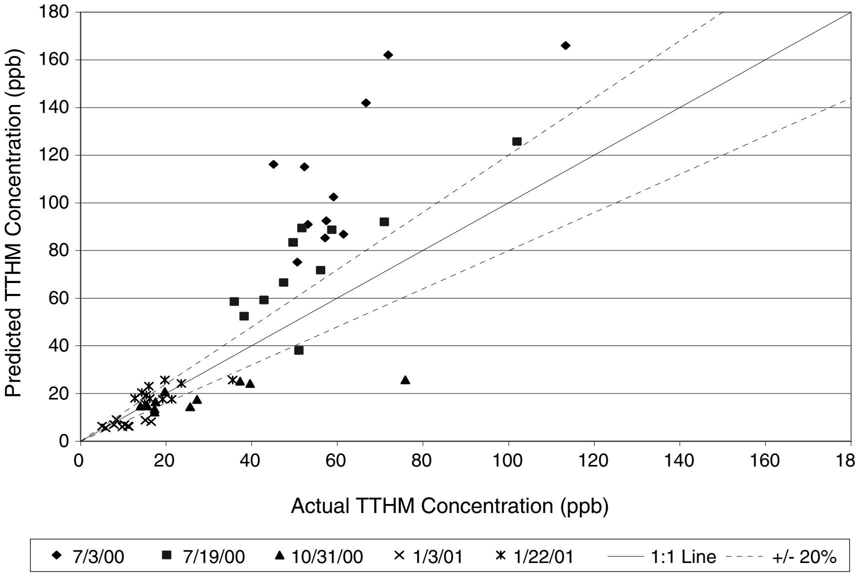

Figure 6 Predicted TTHM concentration versus actual TTHM concentration (all)

summer samples) are above the line, meaning the predicted values were greater than the measured values.

The two sampling dates in July overpredicted the TTHM concentrations on all but one sample site. The three sampling days had much better agreement than the two July sample sets. It is likely that the July sample sets overpredict the TTHM concentrations because the kinetics do not accurately predict their growth. The high initial concentrations led to extremely high limiting concentrations for both sample sets because of the estimated ratio of C_L/C_o. The general trend of the plot is an increase in predicted concentration with increasing measured concentration. This demonstrates that while the model may not have accurately predicted all of the TTHM concentrations, the factors leading to higher TTHM concentrations in the samples also led to higher concentrations in the model (the most important of these factors was the hydraulic residence time).

In order to determine how well the predicted values matched the measured values, the deviation from the measured values was analyzed. Figure 7 presents the relative frequency of various ranges of percent deviation of the predicted values from the measured TTHM concentrations. It can be seen that only about 22% of the predicted concentrations were within ±20% of the actual concentrations, and about 56% of the predicted concentrations were within ±40% of the actual concentrations. A higher percentage of predicted concentrations would need to be within these ranges in order to give confidence in using the model to predict unknown concentrations. Another concern was the number of predicted concentrations that deviate more than 80% from the actual concentrations (almost 8%); almost all of these points were from the two July sample sets.

The model predictions for the HAA9 concentrations had better agreement with the measured HAA9 concentrations than the predicted TTHM modeling. Figure 8 presents a plot of the HAAs versus the actual HAAs for all the sampling points. The solid, straight line on the plot represents a 1:1 relationship. The points are scattered on each side of the line, and there was a fairly even distribution of points that overpredicted and underpredicted the actual concentration.

Figure 9 presents the relative frequency of various ranges of percent deviation of the predicted values from the measured HAA9 concentrations. Almost 70% of the predicted concentrations were within ±20% of the measured values and just over 90% of the predicted concentrations were within ±40% of the measured concentrations. These proportions provide a higher level of confidence in the predictive capabilities of the

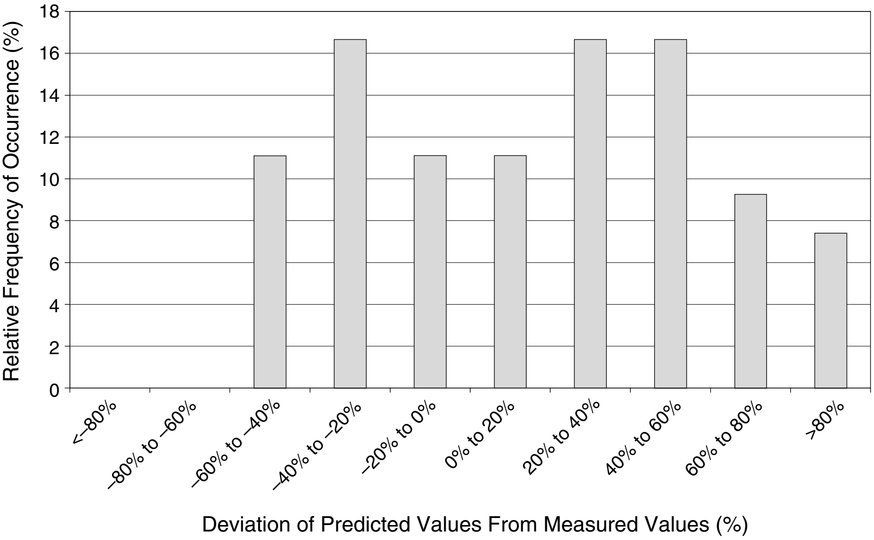

Figure 7 Occurrence of deviations between predicted and measured TTHM concentrations

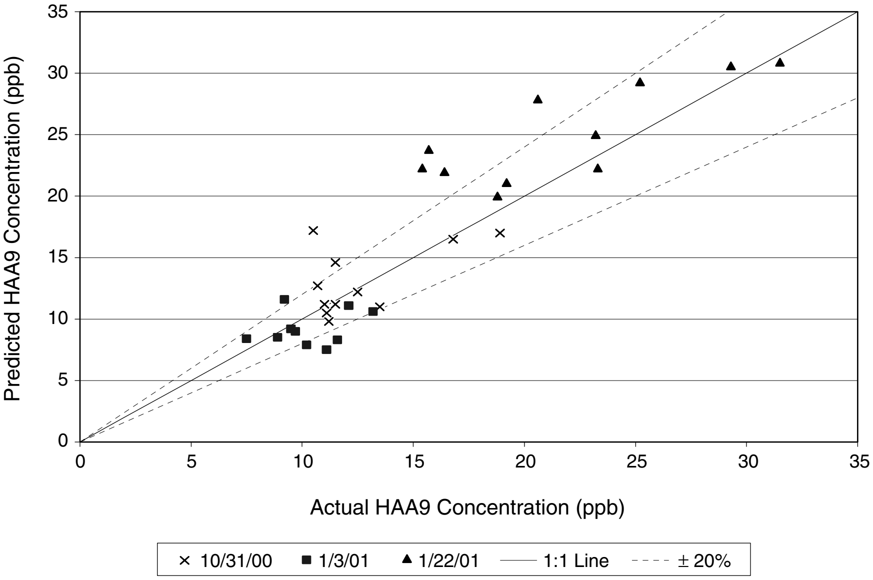

Figure 8 Predicted HAA9 concentration versus actual HAA9 concentration (all)

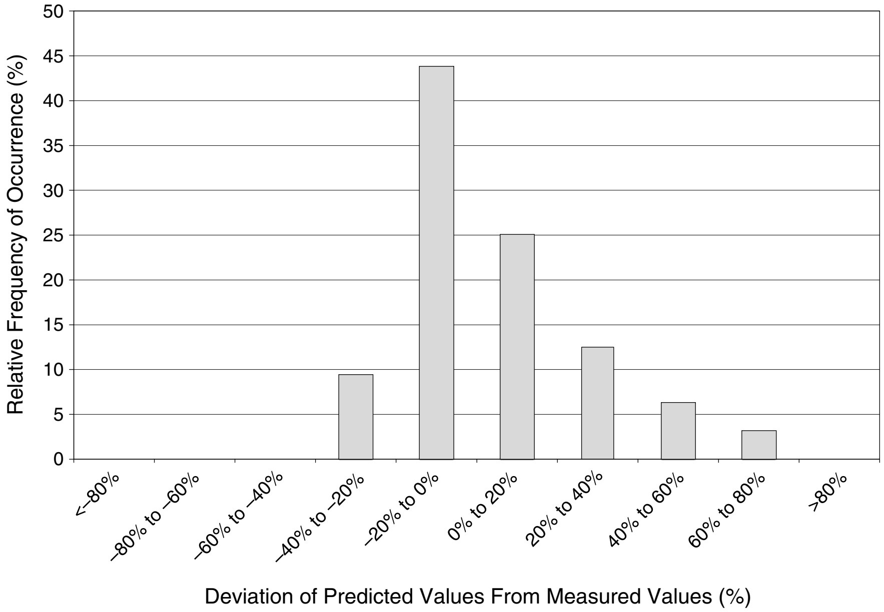

Figure 9 Occurrence of deviations between predicted and measured HAA9 concentrations

model for the HAA9 concentrations. On the other end of the spectrum, there were no predicted concentrations that were greater than ±80% for the measured concentrations. This also gives confidence that the model will not produce results that are unreasonable.

CONCLUSIONS

The distribution system can have a considerable effect on the water quality in drinking water supplies. It has been demonstrated in this study that once the treated water enters the distribution system, there can be a wide variation of several water quality parameters throughout the distribution system. With the goal of providing water that is safe and of high quality to consumers, the distribution system must be continually evaluated and upgraded in order to ensure that the efforts of implementing improved technology at the treatment plant are not wasted. Because of the effect of time on water quality, several locations in the distribution system consistently receive lower-quality water than others. This illustrates the deficiency of regulations that are written based on the average of water quality parameters in the distribution system. Future DBP regulations in which MCLs are based on the maximum locational average are an important step in ensuring that health risks are minimized at all locations throughout the distribution system.

The relationship between the decay of chlorine and the growth of THMs and HAAs was demonstrated in both field and laboratory data, although the relationship was much stronger in the laboratory data. This is because of the chlorine demands in the distribution system, which are not present in the lab. The results of the distribution system modeling met with varying success. The results from the HAA modeling appeared to be more successful than those of the THM modeling. The modeling may not presently be able to consistently provide accurate predictions of expected THM and HAA concentrations in the distribution system; however, the results were promising. It is believed that with further refinement and calibration of the distribution system model and further kinetic studies to better understand the kinetics of both THMs and HAAs, the model could be useful in analyzing the formation of THMs and HAAs in the distribution system. The importance of kinetics on the limiting concentration, estimated by multiplying the initial concentration by the estimated ratio of C_L/C_o, appears to have resulted in large overpredictions of THM concentrations in both of the July 2000 sample sets. The study is site-specific, but the methodology described is quite general and may be applied to other distribution systems.

ACKNOWLEDGMENTS

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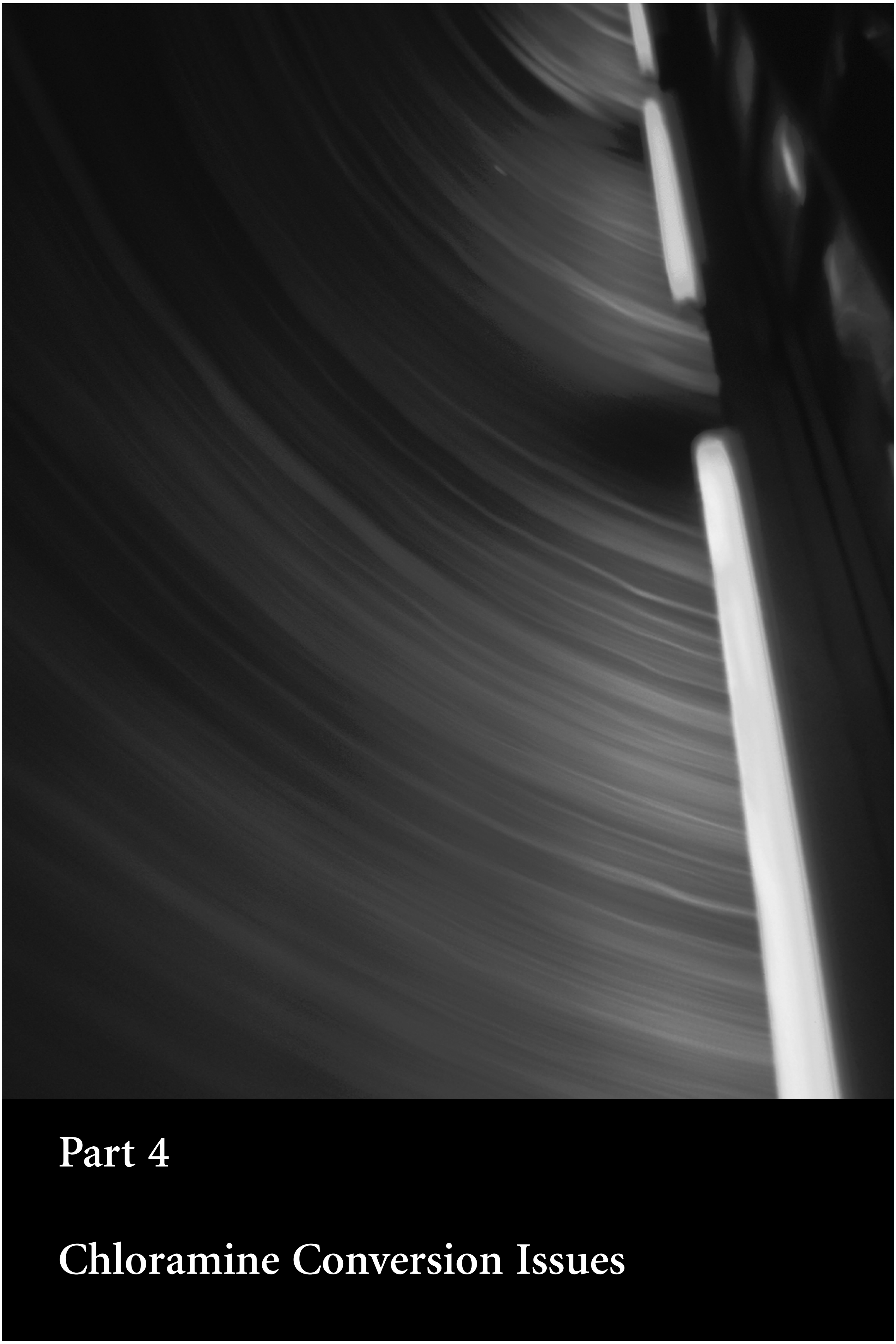

Part 4

Chloramine Conversion Issues

CHAPTER · 13

Blending Chlorinated and Chloraminated Water in Multiple Distribution Systems

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INTRODUCTION

Blending refers to water suppliers mixing multiple supplies either upstream of treatment works, during treatment, or after treatment. The focus of this chapter is blending waters containing different secondary disinfectants. Situations that give rise to water suppliers practicing or considering blending water of different quality include:

- One water supplier purchases water from another, where one supplier modifies disinfection practices to respond to changing regulations.
- Connections to the adjacent system are used in emergency situations.
- Adjacent systems are joined together to manage growth.

Of concern is the ability to manage water quality in the distribution system when distributing water that sometimes contains a free chlorine residual, sometimes a combined chlorine (chloramine) residual, or sometimes a combination or blend of chlorinated and chloraminated water.

If blending cannot be avoided, operating strategies and engineering controls can be implemented to minimize the water quality impacts of blending. Case studies are presented herein to demonstrate the nature of these impacts and to suggest appropriate control measures.

IMPACTS OF BLENDING CHLORINATED AND CHLORAMINATED WATER

An assessment of the feasibility of blending chlorinated and chloraminated water in a system or in multiple systems begins with the application of the theory of breakpoint chlorination. The theory of breakpoint chlorination has been described elsewhere [1–4]. The following issues are noted in reference to Figure 1 (adapted from [3]):

- As the Cl_2:NH_3-N ratio increases to 5:1 on a weight basis, monochloramine is the dominant type of chloramine formed, although trace levels of dichloramine may also be formed, depending on pH.
- When the Cl_2:NH_3-N ratio ranges from 5:1 to 7.6:1, a decrease in chlorine residual can be observed as monochloramine is converted to dichloramine, nitrogen gas, and nitrogen trichloride. Potentially strong odors have been associated with the formation of dichloramine (>0.8 mg/L) and nitrogen trichloride (>0.02 mg/L).

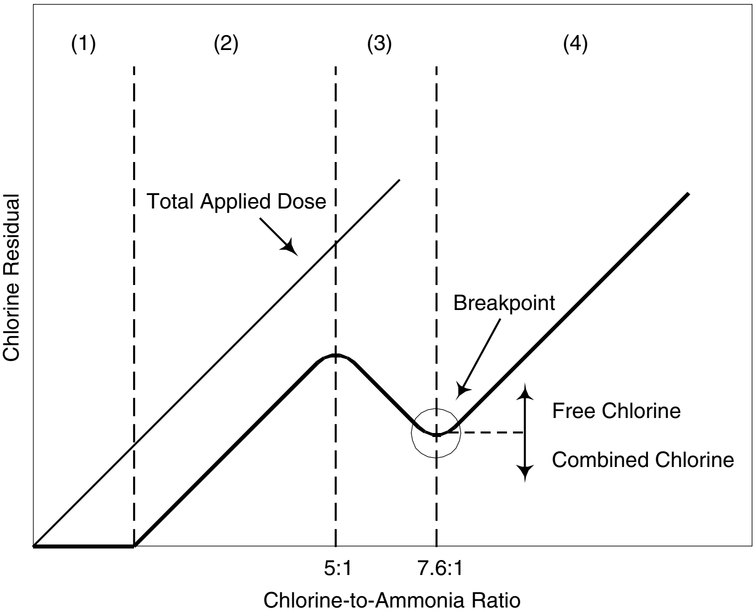

Figure 1 Theoretical breakpoint curve

- The theoretical breakpoint occurs at a Cl_2:NH_3-N ratio of 7.6:1. Operating at the breakpoint may result in minimum chlorine residuals. Chloraminating systems that observe a sudden loss of residual in their system may be operating at breakpoint.

Favorable conditions for nitrification may occur when water is blended among multiple systems. When systems operate at low Cl_2:NH_3-N ratios, the excess ammonia present may be used by ammonia-oxidizing bacteria to form nitrite. However, reports have been made in the literature that even relatively small amounts of ammonia may lead to a potential nitrification episode, depending on water quality conditions [5,6]. Nitrification is undesirable because of its effects on water quality, namely the loss of chlorine residuals, increases in HPC bacteria, and increases in nitrite. Because a chlorine demand is associated with the increase in nitrite, the potential exists for Total Coliform levels to increase [7–11].

In general, more chlorinous tastes and odors may be detected with free chlorine than with combined chlorine and, therefore, a change in the taste of the water would be expected with any conversion from chlorination to chloramination or blending of waters. There are differences in the ability of either disinfectant to mask unwanted odors, and this is related to the comparative oxidizing powers of free chlorine and combined chlorine. As a result, some more sensitive consumers may notice changes when blending is practiced.

Attention should be paid to customers receiving alternating supplies of chlorinated water and chloraminated water. This will have impacts for customers who may be sensitive to or have problems with chloramines, including kidney-dialysis patients, aquarium owners, and pool owners. A public promotion campaign should be conducted to inform customers of the impacts of chloramines and the potential to receive water with two different disinfectants if blending is pursued.

Contact was made with American utilities that blend chlorinated and chloraminated water to review their practices [12–15]. From these discussions and from experiences reported in the literature [16–19], it was clear that operating a distribution system to receive blended water can be a challenge. It is difficult to predict and control the water quality in the resulting blend, as the system moves along the breakpoint curve. Hence, if a water supplier chooses to blend or is unable to avoid blending water with different disinfectants, either the impacts of blending need to be tolerated or operational or engineering controls can be implemented to reduce the impacts.

STRATEGIES FOR BLENDING CHLORINATED AND CHLORAMINATED WATER

The experience of water suppliers that have either considered blending or operate their system with a blend suggests that there are a number of strategies available to operate multiple systems with a blend [12–19]:

1. Operate one system with free chlorine and another system with combined chlorine, recognizing the potential water quality impacts in areas where the water meets and blends; develop operating strategies to minimize any detrimental impacts associated with blending, using:
    a. Controlled blending at a single point in the system (for example at a reservoir or pumping station), to consistently provide water carrying the same residual for distribution.
    b. Controlled blending into a hydraulically isolated section of the distribution system.
    c. Uncontrolled blending, with blending occurring in reservoirs and/or the system.
2. Operate multiple systems with free chlorine.
3. Operate multiple systems with combined chlorine.

The role for the various strategies is reviewed in the following case studies.

CASE STUDY 1: ASSESSING THE ABILITY TO CONTROL WATER QUALITY IMPACTS

In this example, two adjacent systems were joined together via a new reservoir [20]. System A is a conventional treatment plant treating Lake Ontario water and uses chloramines for secondary disinfection. System B is a direct filtration facility also using Lake Ontario water, but uses free chlorine for residual maintenance in the system. The existing facilities are inadequate for year-round protection against *Giardia*; expansion plans are under design to improve the primary disinfection capabilities.

Both systems are owned and operated by a single municipal government, and as such, all strategies listed above were assessed for the operation of the soon-to-be-merged systems. Of concern were the operational challenges associated with a blended water supply: the new reservoir can receive water from both systems and can backfeed the water into both systems. With the commissioning of the new reservoir, the two systems can no longer be hydraulically isolated from each other. Additionally, an increase in customer complaints was expected in areas through which the blending interface would move. As the blending interface could not be easily controlled in response to changing water demands, an increase in complaints could be significant.

Following a detailed cost-benefit analysis that reviewed water quality performance with free chlorine in System B and combined chlorine in System A, historical taste and odor complaints, disinfection by-product formation, capital costs (to convert both systems to a single disinfectant), and operating costs, a recommendation was put forth to avoid blending and to operate both systems with a single disinfectant. In October 1999, System A was converted from combined chlorine to free chlorine in the distribution system as part of a one-year full-scale trial.

CASE STUDY 2: UNCONTROLLED BLENDING

In this example, water quality performance results from an area experiencing uncontrolled blending are presented [21,22]. Two adjacent systems use Lake Ontario as their source water; the responsibility for the water quality in each system lies with a different municipal government. The hydraulics of System C are such that water from System D feeds an area in the northwest corner of System C.

System C:

- Water is treated by direct filtration, with free chlorine for primary disinfection.
- Free chlorine residuals are topped to 0.7 to 1.0 mg/L prior to distribution. Booster chlorination is practiced at five of eight facilities in the system, with target residuals of 1.0 to 1.2 mg/L free chlorine in water leaving the booster facilities.

System D:

- Water is treated by conventional treatment and free chlorine is used for primary disinfection; chloramines are used for residual maintenance in the system.

- The target residual is 1.0 mg/L total chlorine in clearwell effluent; chlorine boosting is employed in the distribution system to increase residuals to 0.8 mg/L total chlorine.
- A chlorine-to-ammonia ratio of 6.8:1 is used.

In January 2000, System C began monitoring two sites in its system that are regularly fed by System D. Nearly one year of results documenting the magnitude and speciation of disinfectant residuals are shown with results for total plate count in Figures 2 and 3 for the two locations (Site ND and Site BV). The disinfectant present in water at both locations varies from water dominated by a combined chlorine residual to water dominated by a free chlorine residual. Occurrences of elevated total plate count were observed at both locations in the summer and early fall of 2000. At Site ND, total plate count was observed to increase when combined chlorine residuals decreased to less than 0.4 mg/L. At Site BV, similar observations were made when combined chlorine residuals decreased to less than 0.2 mg/L. Information such as this can be used to identify action levels to anticipate and control potential problems.

System C continues to monitor water quality at Sites ND and BV, and both systems are addressing the issue.

CASE STUDY 3: CONTROLLED BLENDING WITH ENGINEERING CONTROLS

In this example, efforts to control blending between two systems are discussed [22].

System E sells water to System F. System E uses river water for its source; this is a challenging water rich in organics and downstream of multiple sewage outfalls. Free chlorine is used for primary disinfection at this conventional water treatment plant; chloramines are added prior to distribution. System F receives water from a single supply inlet equipped with a flowmeter and a sodium hypochlorite metering system (to dose approximately 0.75 mg/L free chlorine). Total chlorine levels approaching 0.1 mg/L have been measured in water at the border between Systems E and F; ammonia levels approach 0.3 to 0.4 mg/L as NH_3-N.

In August 1992, the Medical Officer of Health announced a boil water advisory in response to overgrowth in System F’s distribution system. At issue was whether the contamination was due to the source supply (from System E) or due to a problem within System F itself. Total chlorine residuals were observed to decrease in the system, and levels of

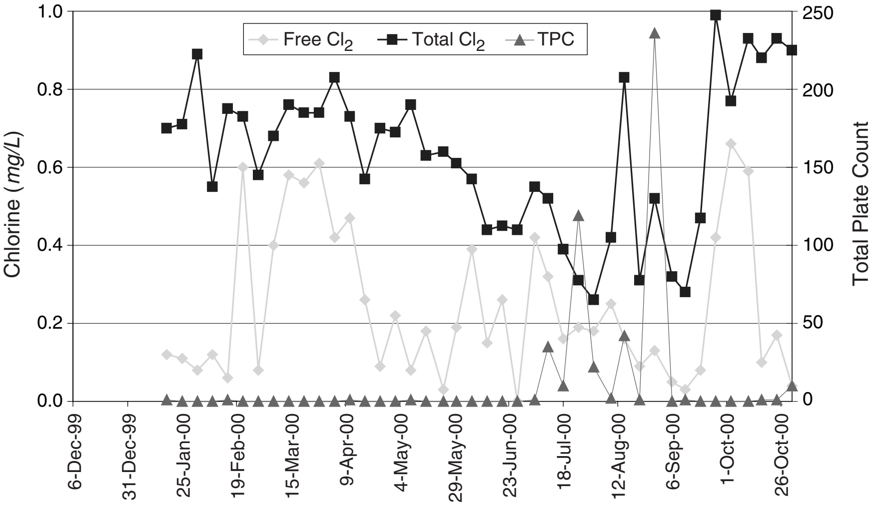

Figure 2 Blending chlorinated water with chloraminated water from another system, site ND

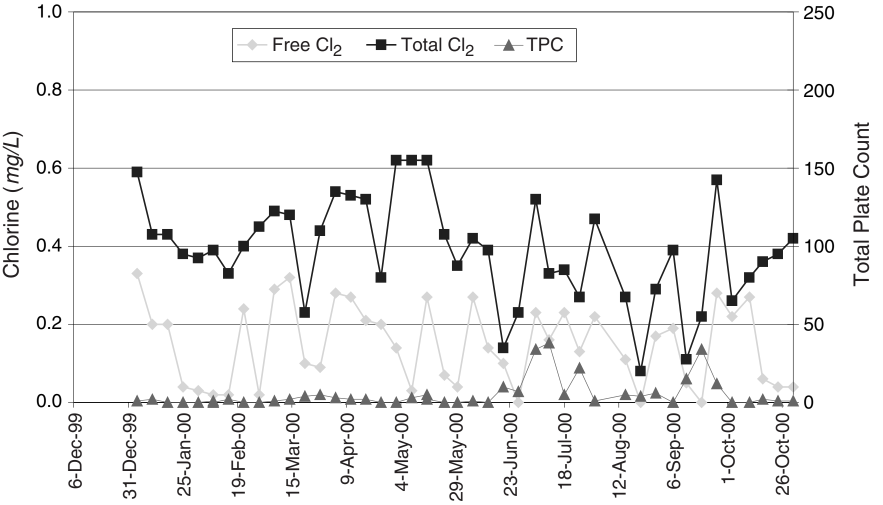

Figure 3 Blending chlorinated water with chloraminated water from another system, site BV

nitrite and heterotrophic plate count were observed to increase. It was suspected that the system was experiencing nitrification.

Historical data from 1983 to 1992 were reviewed and elevated levels of total coliform, total background count, and heterotrophic plate counts were regularly observed from July to as late as January the following year. The travel time in the pipe network alone was estimated to be between 11 and 15 hours. It is noted that treated water is stored in a 1.5 ML elevated tank, approximately halfway between the inlet supply and the end of the system. Based on the theoretical detention time, water remains in this tank for four to five days. Poor mixing in the tank may allow the water to be stored for even longer periods. Detention times in the system were aggravated by the loss of industrial users in the area during the late 1980s and early 1990s.

Chlorine demand and decay studies were conducted at bench scale on water obtained from the supply inlet. Even with doses of 5.5 mg/L, a free chlorine residual could not be detected after 24 hours. The sodium hypochlorite metering pump operated in response to a low-level setpoint in the elevated tank and normally added chlorine to the water entering System F as the elevated tank was filling. At all other times, the chlorination facilities at the supply inlet were not used.

During the outbreak of 1992, the following control strategies were implemented in System F:

- The system was flushed and swabbed to clean the system of bacteriological indicators and to restore disinfectant residuals in the system.
- The elevated tank was shock chlorinated with a dose of 3 mg/L; after positive bacteriological results still occurred, this was later repeated with a dose approaching 9 mg/L.
- The chlorine dose at the supply inlet was raised from 0.75 mg/L to 2.5 mg/L.
- A second chlorination facility was added to the elevated tank, to allow water to be chlorinated at both the inlet and outlet of the tank.
- Breakpoint chlorination was implemented to ensure the presence of a free chlorine residual across System F.

With these operational approaches and structural changes, System F now successfully operates with water supplied from System E.

RECOMMENDATIONS AND CONCLUSIONS

If possible, blending water with different disinfectants should be avoided. If blending is unavoidable, rely on operating strategies and engineering controls to mitigate the impacts associated with blending. Develop a comprehensive monitoring program for the distribution system to delineate potential problem areas and to identify action levels in anticipation of potential problems.

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CHAPTER · 14

Operation and Maintenance of Water Distribution Systems During and After Conversion to Chloramine Disinfection

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Special Acknowledgment: Mid-Peninsula Water District

INTRODUCTION

In the state of California, as well as across the US, many water distribution systems maintain a disinfectant residual through the use of chlorine. However, chlorine can form harmful disinfection by-products (DBPs), which have come under scrutiny from regulatory agencies in recent years. One group of DBPs in particular is trihalomethanes (THMs), which are regulated by the Environmental Protection Agency (USEPA) and the California Department of Health Services (DHS). The original limit of 100 μg/L, set by the 1990 Surface Water Treatment Rule (SWTR), was lowered to 80 μg/L by the 1998 Disinfectants–Disinfection By-products Rule (D–DBPR).

In response to the proposed and existing DBP regulations, some water systems are changing their primary disinfectant from chlorine to chloramines. Chloramines, a combination of chlorine and ammonia, provide reliable disinfection but form no known DBPs. However, if a sufficient chloramine residual is not maintained, issues can arise for distribution systems, such as the ability to meet the Total Coliform Rule and to minimize taste and odor complaints. If a residual is not maintained in the distribution system, the potential for nitrification is increased, which can lead to higher coliform levels and taste and odor issues. Therefore, reducing the nitrification potential of a chloraminated system is crucial.

One main factor in the nitrification potential of a distribution system is the water age. If water is stored for long periods of time within a system, the chlorine within the chloramines will dissipate, leaving behind ammonia, which will promote microorganism growth. Therefore, maintaining low water ages in the system is a critical factor in controlling the nitrification potential and maintaining good water quality.

A chloramine conversion study was performed for the Mid-Peninsula Water District (MPWD) located in Belmont, Calif., which assessed how a conversion from chlorine to chloramines would impact its water distribution system. Three main system components were analyzed in the study: distribution system materials, system maintenance, and system design and operation. Using MPWD as a case study, this chapter presents useful operational, maintenance, and system design concepts for systems that have or are considering a conversion to chloramine disinfection.

NITRIFICATION

As stated previously, a primary concern for distribution systems considering a conversion to chloramine disinfection is the nitrification potential of the system. Nitrification, a stepped biological process, occurs when ammonia is converted to nitrite and then to nitrate by nitrifying bacteria.

$$\text{Ammonia } (NH_3) \rightarrow \text{Nitrite } (NO_2) \rightarrow \text{Nitrate } (NO_3)$$

One indication of nitrification is a rapid depletion of chloramine residual. This depletion may then be followed by other adverse affects:

- Large increases in heterotrophic bacteria
- Increases in nitrite and nitrate concentrations
- Decrease in dissolved oxygen concentrations
- Decrease in alkalinity (Kirmeyer et al., 1995)

The SWTR requires a detectable disinfectant residual throughout the entire distribution system. If the heterotrophic plate counts (HPCs) are lower than 500/mL throughout the system, then a detectable disinfectant residual is assumed. A loss of chloramine residual through nitrification could lead to increases in HPCs, which would cause a lack of regulatory compliance. An increase in HPCs could also cause coliform growth (a waterborne pathogen). Therefore, understanding where and how nitrification occurs is vital for proper management of a chloraminated distribution system to produce good-quality water and stay in regulatory compliance.

PROJECT BACKGROUND

This study of MPWD's distribution system was instituted in anticipation of the San Francisco Public Utilities Commission (SFPUC) switch to chloramines as a disinfectant. The SFPUC owns and operates an aqueduct system that transports water from reservoirs in the Sierra Nevada Mountains to the San Francisco Bay Area. MPWD purchases its water from SFPUC and therefore would be affected by this change. Figure 1 illustrates the location of the MPWD system in relation to the SFPUC aqueducts.

As discussed previously, one purpose for the SFPUC's conversion to chloramines is to reduce the concentration of chlorine-related DBPs in the water. Also, chloramines are more stable than chlorine, which can help to maintain a disinfectant residual in the more distant portions of the distribution system. The planned implementation date for the chloramine conversion is spring 2003.

In anticipation of this conversion, the scope of this project for MPWD included assessing the impacts that the SFPUC's planned conversion to chloramine disinfection would have on MPWD's distribution system. This assessment was accomplished through hydraulic modeling of the system operation using MPWD's updated H2ONet model and by analyzing system materials and design for possible deficiencies.

DISTRIBUTION SYSTEM LAYOUT

Figure 2 conceptually illustrates the MPWD distribution system (for simplicity, not all of the components are shown). Water from the SFPUC aqueducts enters the MPWD system at two locations: through pumping from MPWD's Tunnel Pump Station into the highest reservoirs in the system, and by gravity flow from the Hillcrest Meters. In order to control water pressure throughout the system to within an acceptable range, the distribution system network is divided into nine pressure zones. Nine aboveground reservoirs are used for water storage and supply. The reservoirs have an approximate total volume of 9.0 million gallons (MG). To regulate the pressure and flow between pressure zones, the system contains 6 pump stations, 13 pressure regulating stations, and 2 hydropneumatic stations. The distribution system contains water mains ranging in size from 4 inches to 24 inches in diameter.

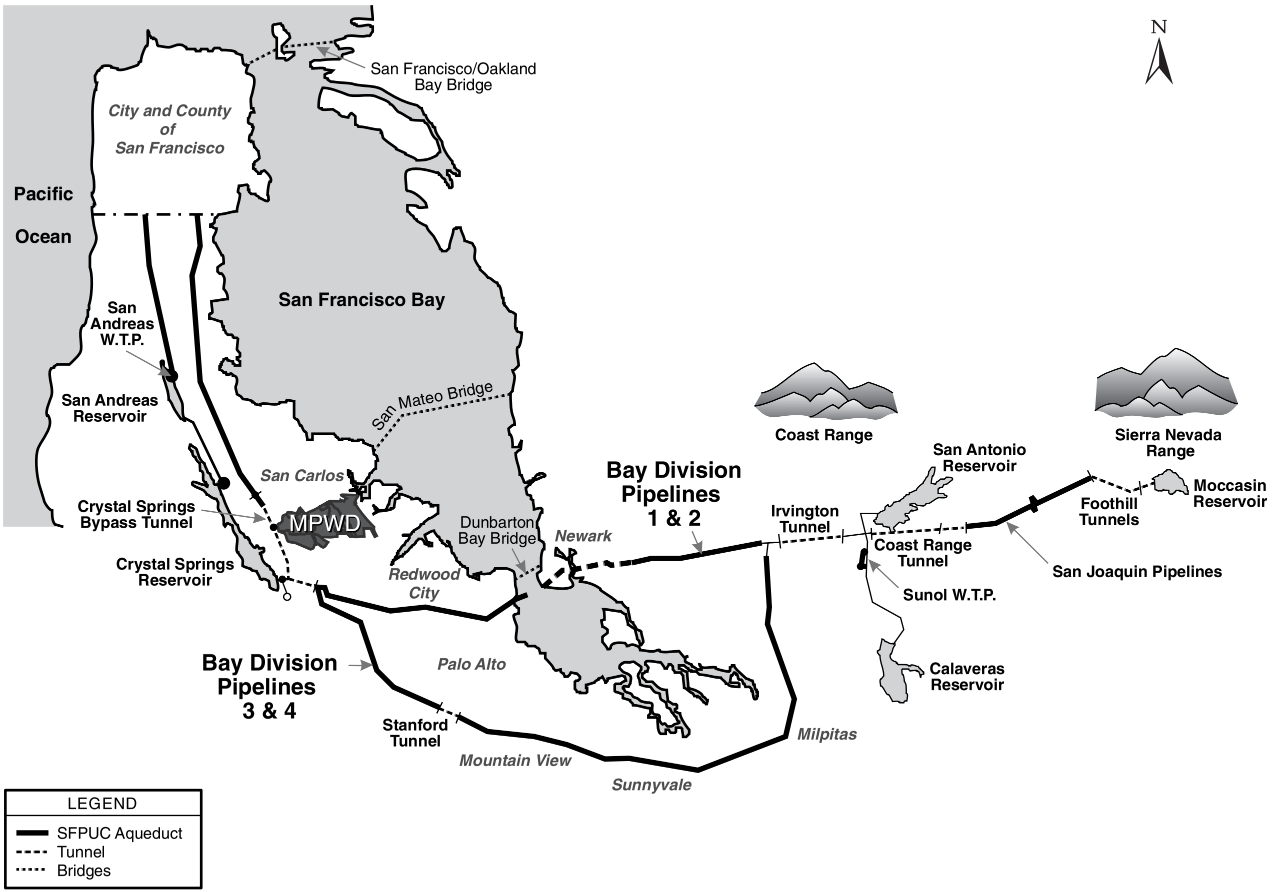


Figure 1 SFPUC water system layout

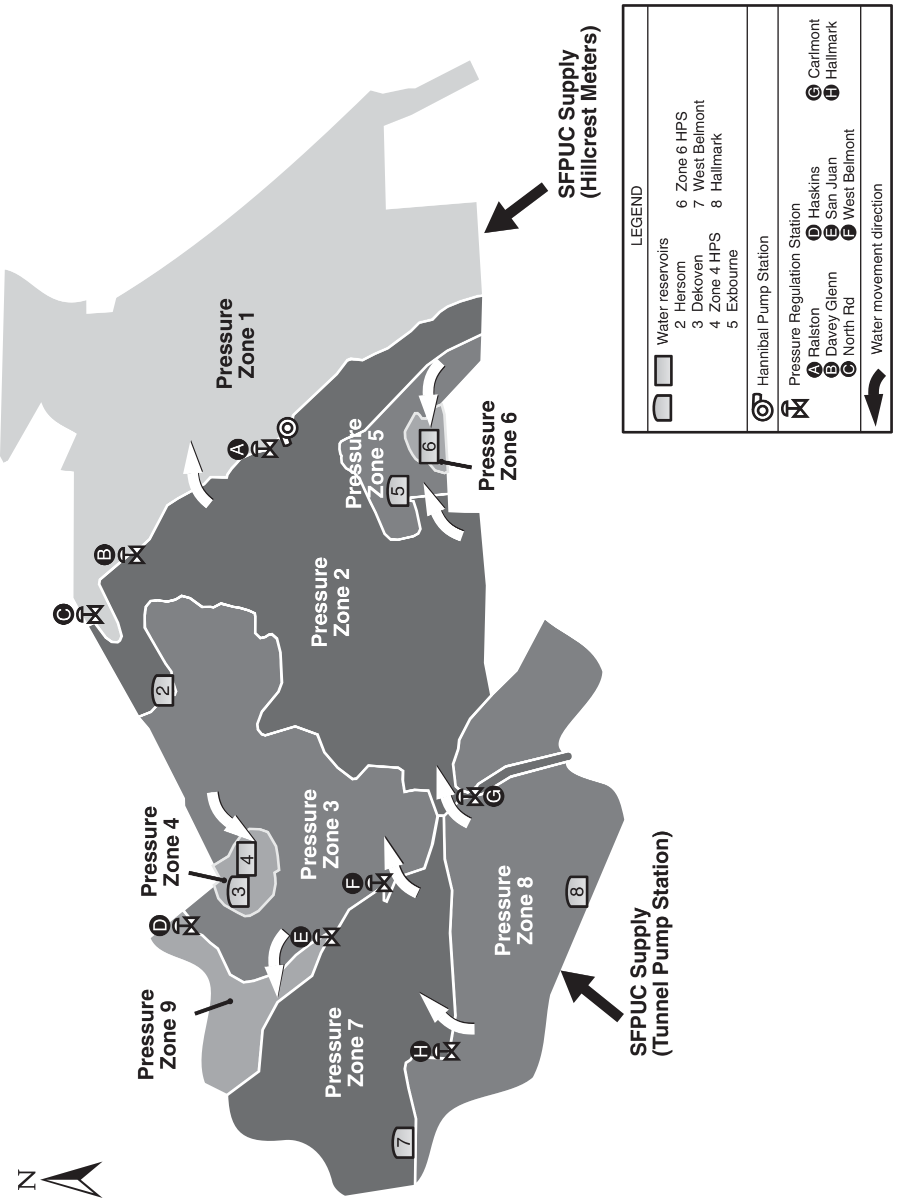


Figure 2 MPWD normal operation schematic

CHLORAMINE CONVERSION ANALYSIS

Chloramines present concerns regarding the potential for increased distribution system component failure and water quality problems such as nitrification and positive bacterial counts. An analysis was performed on MPWD's system that reviewed its system materials, design, and operation to identify potential areas of concern with respect to the upcoming chloramine conversion. Specifically, the issues addressed were:

- Distribution system materials' susceptibility to chloramine attack
- Design and operation changes to help prevent nitrification
- Maintenance changes to preserve water quality

The analysis of these issues will be addressed in the following sections.

Distribution System Materials

MPWD's distribution system components considered in this evaluation were pipelines, valves, fittings, and elastomeric components (seals, gaskets, O-rings, etc.). Distribution system materials including piping, fittings, valves, and other appurtenances are typically constructed in accordance with ANSI/AWWA standards. The ANSI/AWWA standards were used in this study to evaluate materials within the distribution system that may be susceptible to degradation after the switch from chlorine to chloramines.

Pipeline materials

The MPWD distribution system includes piping made of cast iron (CIP), asbestos cement (ACP), ductile iron (DIP), concrete (CCP), polyvinyl chloride (PVC), polyethylene (PE), wrapped steel, and, in small-diameter piping and tubing, copper and brass. Similarly, distribution system valves and fittings are constructed of cast iron, bronze, stainless steel, brass, and other similar materials. It has not been shown that chloramines corrode most of these materials any more quickly than does chlorine. Thus, for the most part, these piping materials will not be affected by the conversion and MPWD will not experience any increased failure of them due to the conversion.

There has been some evidence, however, that copper and brass may be more susceptible to corrosion due to chloramines. The research results, however, are not conclusive. Reiber (1993) reports that chloramines may actually pose a smaller corrosion threat to copper and brass than chlorine. Therefore, piping material degradation was not considered to be a big threat to MPWD's system. Because available research literature did indicate a primary concern is elastomer degradation, the remainder of the evaluation focused on the elastomers.

Elastomers

ELASTOMER TYPES AND USES. Elastomers are manufactured from one or more base polymers that are compounded with several other ingredients, including fillers, plasticizers, antidegradants, accelerators, and vulcanizing agents (Reiber, 1993). Fillers may be used to extend the quantity of the finished product or to alter the hardness and smoothness. Plasticizers are used to increase the low temperature flexibility or to reduce the hardness of the finished material. Antidegradants are added to the elastomer to improve resistance to chemical attack. Accelerators enhance the kinetics of the vulcanization process. Vulcanizing agents, such as sulfur, organic peroxides, and metallic oxides, are responsible for the cross-linking of the base polymer to produce a strong elastomer. Table 1 presents elastomer types commonly used in water distribution systems.

ELASTOMERIC DEGRADATION AND FAILURE. Prior to the 1993 AwwaRF study (Reiber, 1993), data indicated the most common elastomeric failures in chloraminated systems were found to occur in domestic applications such as rubber parts in toilet tanks. Since 1993, anecdotal information from various utilities across the country has confirmed this finding (Reiber, 1999; LeChavellier, 1999). Reiber (1993) found examples of other reported failure incidents, including gaskets and diaphragms on backflow prevention devices, O-rings in poultry watering devices, and aerator washers in airport bathrooms.

Elastomers were found to degrade or fail from two possible oxidative attack mechanisms caused by either chlorine or chloramines (Reiber, 1993). These two mechanisms are: (1) attack on the polymeric backbone making up the elastomeric structure and (2) attack on the cross-linking between polymer groups.

Cross-linking of polymer groups occurs during a process termed vulcanization. Research has shown that it is generally the cross-linking property of the polymer that makes it more susceptible to oxidative attack (Briggs et al., 1963). The seven base polymers, shown in Table 1, are commonly used in water distribution systems and often have cross-linking properties. Therefore, it is important not only to know the

Table 1 Elastomer uses in distribution systems

Polymer Type	Usage
Natural rubber (NR)	Flapper valve, large pipeline gaskets
Synthetic rubber (SBR)	Gasket material
Butyl rubber (IIR)	Diaphragms
Neoprene (CR)	Pumping impellers, valves seats, chemical feed pumps
Ethylene-propylene-diene (EPDM)	O-rings, valve seats, flat gaskets
Nitrile (NBR, buna-N)	O-rings, valve seats, pump impellers, check balls
Silastic (SI)	High-temperature applications

NOTE: Letters in parentheses after polymer type are rubber industry abbreviations.

Source: Reiber, S. (1993)

internal oxidative attack mechanism but also to understand how elastomers are exposed to disinfectants within the system.

One way in which elastomers are exposed to disinfectants is simply by surface contact. Surface deterioration of the elastomer has been found to be accelerated by chloramine exposure (Reiber, 1993). Surface deterioration can lead to failures caused by surface cracking, hardness of material, permanent surface distortion, or loss of filler material. Chlorine and chloramine attack at the surface of elastomers, which then causes breakdown of the material surface, allowing water absorption into the polymeric structure of the material. The absorption of water into the polymer structure leads to the swelling of the elastomer. The swelling correlates with deterioration of performance (stress and strain resistance). Reiber (1993) showed in accelerated life-cycle testing that chloramine caused a greater degree of water absorption, in some cases yielding a weight gain of 70%.

Reiber (1993) researched elastomer exposure degradation by conducting accelerated life-cycle testing at elevated temperatures (i.e., 100°F) and concentrations (i.e., 300 mg/L of hypochlorous, hypochlorite, dichloramine, or monochloramine solution). Based on this research, Reiber determined the susceptibility of elastomers to several types of failure. The physical effects that were considered include: (1) surface cracking, (2) hardness (embrittlement), (3) permanent surface distortion,

(4) surface tack (caused by plasticizer leaching), and (5) loss of filler material.

From the results of the study, it is apparent that elastomers vary in their susceptibility to both free chlorine and chloramine. In general, hypochlorous acid and dichloramine (pH 5.5) exposure caused more severe deterioration than the hypochlorite anion and monochloramine (pH 8.5). One elastomer of particular concern, especially for the MPWD system, was styrene-butadiene (SBR) as it is specified by ANSI/AWWA C111/A21.11-90. The results showed that SBR was much more susceptible to chloramines than it was to chlorine. This is pertinent since it is likely the most common material specification in the MPWD system. This and other results of the Reiber (1993) study were used to determine the likely effects of chloramines on elastomers in the MPWD system.

SPECIFIC CONSIDERATIONS FOR MPWD. The average pH of SFPUC water is about 9. Based on this pH, it is likely that the free chlorine presently in the MPWD system is in the hypochlorite anion form. However, upon changing over to chloramine disinfection, assuming the chlorine–to–ammonia-nitrogen ratio is less than or equal to 5:1 and the pH is 9, it is likely that chlorine will occur as monochloramine, as desired. In Reiber's (1993) work, however, the conclusion was made that the severity of attack and degradation on elastomers from both chlorine and chloramine followed the order:

dichloramine>>monochloramine>hypochlorous acid>hypochlorite anion

Therefore, the change to chloramine will perhaps increase the rate of elastomer degradation. However, since Reiber's accelerated life-cycle testing was conducted at elevated temperatures and concentrations, the results may be more of a worst-case condition.

MPWD's average operating temperature, disinfectant residual, and pH are far less extreme than those used in the accelerated life-cycle testing. In fact, MPWD's average temperature likely ranges from 50°F to 75°F; average water pH in the system is about 9, and disinfectant residual level is below 1.5 mg/L. Though dramatic or rapid failure of elastomeric components is not anticipated, Table 2 provides a summary of those parts of MPWD's distribution system where degradation rates may increase due to the change to chloramine disinfection. The factors considered in developing Table 2 include:

- The potential for continual water exposure versus exposure to stagnant ("dead") water

- The added physical/mechanical action of valves opening/closing
- The relative frequency of exposure

Table 2 Summary of potential problem components in MPWD distribution system

Component	Material Likely to Be Present	Potential for Increased Degradation
Flange gaskets	Red rubber (SBR) and elastomeric binders suitable for potable water	Moderate
PVC joints/gaskets	Material compatible with plastic material*	Moderate
Hydrant valves & gaskets	Vulcanized SBR	Moderate
DIP push-on joints	Vulcanized SBR	Low
Valves seats (various)	Vulcanized SBR	Low to moderate
Mechanical joints	Vulcanized SBR	Low
Storage tanks	Vulcanized SBR	Low to moderate
Toilet tank flapper gaskets/float balls†	SBR	Moderate
Water heater valves‡	Celcon® or Delrin®	Low to moderate

*Typical for pipe distributors to provide SBR gaskets

†Newer toilet tank flapper gaskets made with rubber manufactured with antidegradants that prolong the rubber life

‡Liners and valves may be manufactured with thermoplastics.

NOTE: Ranking encompasses physical and/or operational considerations, in addition to the potential degradation from exposure to chlorine and chloramine. For hydrant valves and other valve seats, it is assumed that most of the time, exposure will be fairly static, meaning little continual water flow will occur. However, because these components will incur stresses due to operational opening and closing, any potential degradation is increased by this physical action. In water storage tanks, any gasket(s) on the inlet or outlet will experience continual exposure.

Summary of distribution system materials review

Based on the analysis presented above, it was not anticipated that MPWD would experience any rapidly increased failure rates of distribution system components. Rather there may be some moderately increased rate of degradation but, given the variability in elastomer formulations, water quality effects, and mechanical stresses, it was not possible to effectively identify any portion of the system that requires replacement prior to conversion. Therefore, MPWD was advised to:

- Monitor water losses in the distribution system either through leaking hydrants and seals or through unaccounted-for water losses. When the unaccounted-for water losses show an increase, a detailed water audit or leak detection program may be warranted.
- Maintain a replacement supply of elastomeric components that are made of more chloramine-resistant material.

System Design and Operation

System design and operation can play a large role in the water quality of any distribution system. These parameters become particularly important when a system's disinfectant is converted from chlorine to chloramines. A primary concern with chloramines is the decay of the chlorine residual within the system. Several factors can affect chlorine decay, such as temperature, the degree of mixing within the reservoir, detention time in the reservoir, and sediment and biofilm presence in the reservoir.

Thus, controlling water age in the distribution system and the reservoirs, in particular, is important in maintaining water quality in a chloraminated system. The more time the water spends in the reservoirs and the system, the lower the chlorine residual and consequently the more ammonia that would potentially be in the water. Ammonia in the reservoirs could lead to microorganism growth and nitrites in the water, which would be detrimental to water quality. Thus, water age was the main focus for this study.

Water demand and water age calculations

The system water ages were determined using the city's updated, calibrated H2ONet hydraulic model. The updated H2ONet model was able to run an extended period simulation (EPS), resulting in pressure and flow estimates over a selected period of time. Due to the time period simulation, diurnal water demands were developed for the system using both yearly and seasonal SCADA information. This information was then used in the model to calculate water age in the system.

The seasonal demand trends were used to calculate the magnitude of the maximum and minimum demands with respect to the average daily demand. Once this was complete, the maximum, average, and minimum diurnal curves were developed and used in the model. Figure 3 illustrates the three diurnal curves that were used.

Using the diurnal demand information, three water age scenarios were examined during the minimum, average, and maximum day demand conditions. Of the three scenarios, the minimum water demand was expected to produce the highest water age due to the low demand on the system and the reservoirs. Average day and maximum day demand were expected to have moderate to good water age. The water age goal throughout the system was approximately 16 days (or 400 hours). This corresponds to about half the age (30 days) determined by the SFPUC lab tests at which the chloramine residual decays significantly. The residual loss could then lead to nitrification. The 400-hour goal would provide MPWD with a factor of safety to account for conditions not studied in the lab tests. According to the SFPUC, the water entering the MPWD system is about 1 day old.

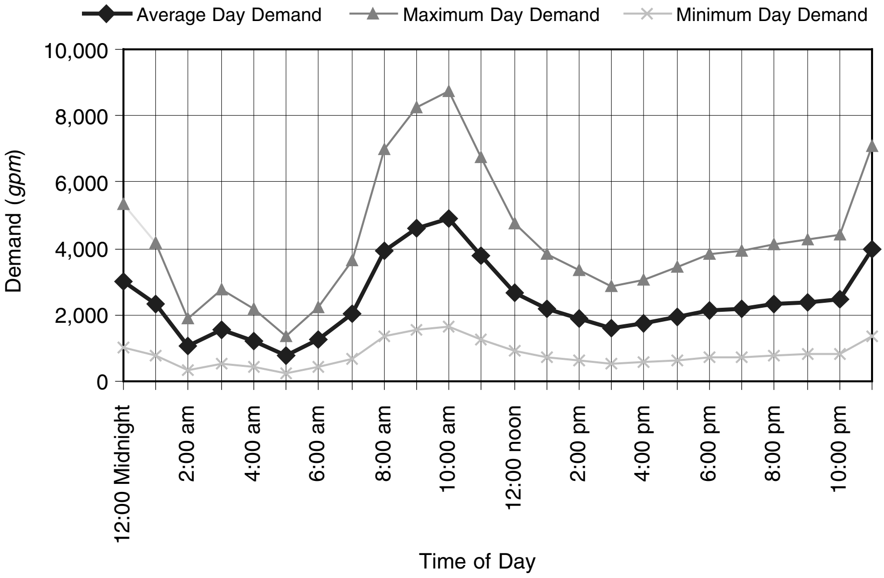


Figure 3 Average, maximum, and minimum diurnal demand curves for MPWD's system

Current system operation

MPWD's current method of distribution system operation is in a "feed-down" mode. In other words, much of the system's water enters at a high elevation and is allowed to gravity-flow through the system with the help of a series of reservoirs and pressure-regulating valves. Figure 2 illustrates graphically the movement of water throughout MPWD's system.

Water enters the system in two locations: Tunnel Pump Station and Hillcrest Meters. Tunnel Pump Station pumps water into the Hallmark Reservoirs (one of the highest points in the system), located in Pressure Zone 8. From Pressure Zone 8, water moves into Zone 7 and the West Belmont Reservoir through the Hallmark Regulating Station. From Zone 7, water is gravity-fed to Zone 3 and the Dekoven Reservoirs by way of the West Belmont Pressure Regulators. Once in Zone 3, water can move by gravity into Zone 9 through the Haskins or San Juan Regulators. Water from Zone 3 is also pumped into the Zone 4 Hydropneumatic Station (HPS), which provides for Zone 4 water demand. Pressure Zone 8 also provides Zone 2 and the Hersom and Exbourne Reservoirs with water through the Carlmont Regulator.

Water can also be pumped into Zone 2 from Zone 1 through the Hannibal Pump Station, using either a small (50 hp) or large (150 hp) pump. The smaller pump is used during winter operation, while the larger pump is needed during summer demand periods. The main source of water for Pressure Zone 1 is the Hillcrest Meters. Three pressure regulators, Davey Glen, Ralston, and North Road, can also supply Zone 1 with water from Zone 2 during emergency or extremely high water demand conditions (such as during firefighting).

Minimum day demand

A preliminary review of the water ages in the system under minimum day demand conditions assuming the current system operation showed water ages much greater than the 400-hr goal. Therefore, a modified system operation was determined that resulted in lower overall water ages.

This new, modeled operation consisted of limiting the supply from the Hillcrest Meter turnout and taking water into the system only through Tunnel Pump Station. However, turning off the Hillcrest Meters during the entire minimum demand period could lead to high water ages along the 4-mi pipeline. Flushing of the pipeline once per week for 8 hr was recommended to maintain low water age and good water quality.

By limiting the flow from the Hillcrest Meters in this manner, the system was forced to operate in a true "feed-down" operation, using the pressure-regulating stations to provide the majority of the water from Pressure Zone 8 down to Zone 1. All zones operated under the same conditions, except that now Zone 1 was delivered water by Zone 2 through pressure regulators. The main advantage to this mode of operation was that water moved throughout the entire system more rapidly. Another critical control parameter was that all nine reservoirs were allowed to drain to 60% of their total height before being refilled. This also allowed for more turnover in the reservoirs and quicker movement of water in the system, which resulted in lower water age.

Figure 4 is a graphical representation of the recommended system operation during minimum day demand.

Average day demand

Again, using MPWD's current method of operation, the model was used to calculate water age within the system during average day demand. A review of the results indicated that the water age in the system exceeded the 400-hr limit in certain areas of the system. Therefore, a slight modification to the current method of operation was modeled, which resulted in most of the water age throughout the system meeting the 400-hr criterion.

In this suggested modification to the normal operating parameters, reservoir tank level was the main focus. To maintain water ages within the specified limit, the Hallmark, West Belmont, and Dekoven tanks would be required to drain to at least 70% of their full level before being refilled. The Hersom, Exbourne, and Buckland tanks were drained to 60% of their full levels before being refilled. The lower tank levels were modeled and the result indicated that this change allowed for more turnover in the reservoirs and lower water ages. Also, the modeling indicated that the Hannibal Pump Station should be operated intermittently so that the Zone 2 reservoirs, Exbourne and Hersom, would drain and refill more frequently, keeping water ages low.

Under this operating scheme, the main operational change throughout the system would be resetting the reservoir levels, allowing the tanks to operate at their respective levels.

Maximum day demand

Because of the large demand placed on the system, it was not expected that the maximum day demand scenario would produce high water ages

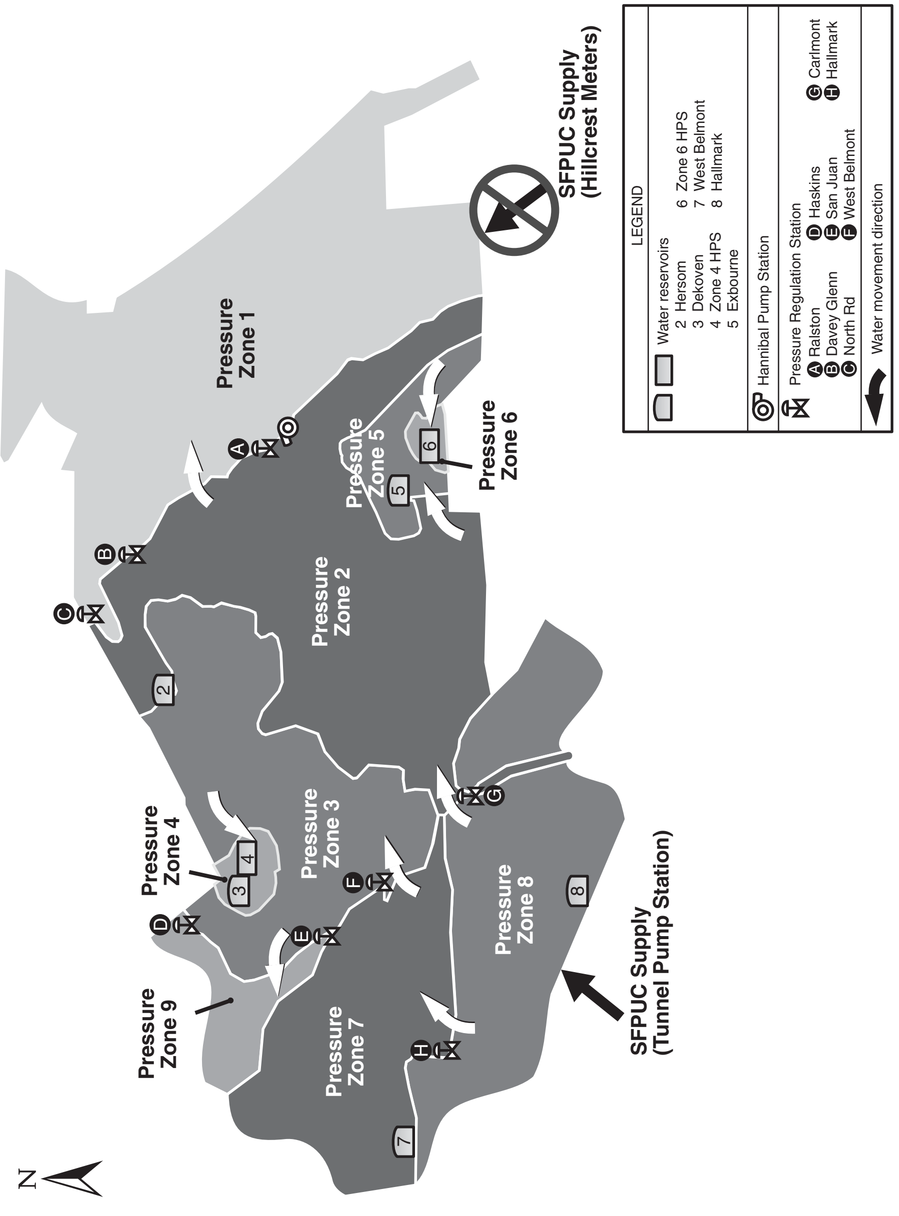


Figure 4 Recommended modified operation during minimum day demand conditions

even in the reservoirs. The water age modeling showed similar results, as expected. The model calculated water ages in the majority of the system that were within the limit of 400 hours.

Other considerations

DEAD-END MAINS. Even though the majority of the areas within the system showed water ages within the 400-hr limit, there were a few dead-end mains that remained problematic for all three scenarios. The dead-end mains usually had very little demand associated with them and therefore, would age very quickly. These mains would need to be either looped back into the system or flushed on a regular basis to ensure good water quality.

STORAGE FACILITY DESIGN. It is important to note that the model assumed that the reservoirs in the distribution system were perfectly mixed at all times. The predicted water ages in the above scenarios were dependent on the reservoirs being adequately mixed. In order to ensure adequate mixing, water turnover rates (reducing water age) and reservoir inlet/outlet dynamics were examined. These key factors would reduce the potential for nitrification in the system.

Since methods of reducing water age throughout the system have already been established and discussed, the issue of reservoir inlet/outlet dynamics was addressed. All reservoirs in the MPWD system were aboveground and had a common inlet/outlet configuration, as shown in Figure 5. This type of configuration leads to low turnover in the reservoir. Water will continue to age near the top of the tank while water near the inlet/outlet will be constantly drawn out and refilled. Also, aboveground reservoirs have a high potential for temperature stratification. Influent water temperatures are typically lower than the ambient temperature of the tank, and the influent flow will tend to sink. High temperatures in the water in combination with high water ages near the top of the tank could result in nitrification problems.

To avoid stratification within the reservoirs, a modification to the inlet/outlet configuration was recommended. One alternative to be considered was a single riser pipe inlet and outlet configuration, which would promote better mixing. Figure 6 shows a typical piping plan that would require minimal modification on the inside of the reservoir only. Piping extension would be needed to create a separate inlet and outlet port and a check valve would control the flow into and out of each pipe. A second alternative to induce mixing in the tank entailed installing a

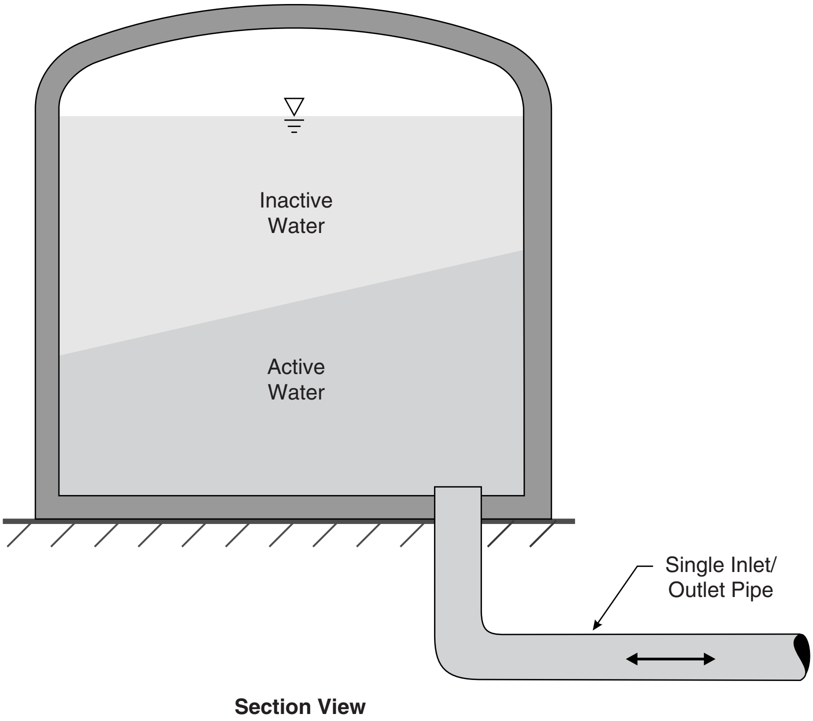
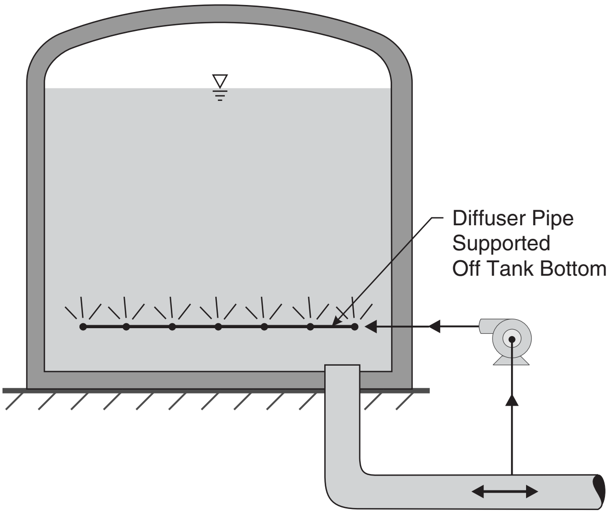


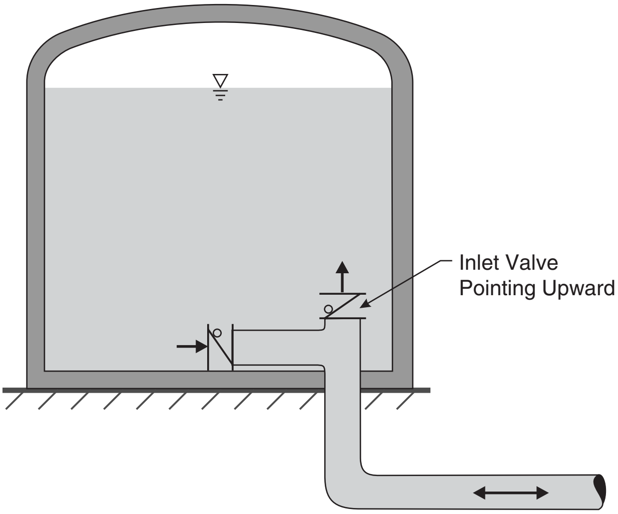
Figure 5 Existing reservoir configuration

pumped recirculation system in the bottom of the tank. Diffuser piping, supported off the bottom of the tank, and a small pump (generally, 1 hp/MG) would improve the mixing conditions in the reservoir. Diffuser piping would need to be designed for each tank configuration to ensure a well-mixed tank. Figure 6 also illustrates an example of a recirculation system.

OPERATIONAL TIMING. As shown previously, water age in the system also depends on the mode of operation, and the operation is dependent on the demand period. Therefore, it was necessary to estimate when these three demand periods would occur throughout the year for the MPWD system. Based on the seasonal data provided by MPWD for the past three years, the minimum day demand period occurred from November to March. Maximum day demand conditions usually began in May and lasted until late August. Average demand conditions could be assumed throughout the remainder of the year. As stated, though, this was only an



Section View



Section View

Figure 6 Reservoir retrofit: (A) pump mixing and (B) separate inlet/outlet

estimate based on previous seasonal patterns. Maximum conditions could occur as early as March and last until late October. Therefore, it will be important for MPWD to track the demand trends throughout the year and adjust system operation as necessary to meet the system demands.

System Maintenance

Distribution system maintenance can be the biggest defense against the nitrification potential in a chloraminated system. System cleaning, main flushing, reservoir cleaning, and water quality monitoring can all serve toward keeping bacteria out of the system, which will reduce the likelihood of water quality problems. Methods of maintenance on a chloraminated distribution system are discussed in the sections that follow.

System cleaning

As described in the previous sections, one key factor in preventing nitrification from occurring in the distribution system is preventing the chloramine residual from degrading. This is best done by keeping the water age low and maintaining a clean water distribution system. Chloramine will degrade when it is in contact with organic material it can oxidize. Typical organic materials of concern include:

- Total organic carbon, turbidity, and other chemical constituents in the water
- Iron tuberculation and encrustation in the pipe walls
- Biofilms in the pipe and reservoir walls
- Deposited sediments in pipelines and reservoirs

The SFPUC water is of relatively high quality, meaning that it is low in total organic carbon, turbidity, and other chemical constituents that could cause the chloramine residual to degrade prematurely. This was evidenced by the SFPUC's own tests that indicated the water can be stored for up to 30 days before the chloramine residual degrades significantly (Camp, Dresser and McKee, 1999). These tests, however, were conducted under laboratory conditions. Though the water temperature was varied to simulate conditions that might be realized in the distribution system reservoirs, the water was held in clean amber bottles. Thus, the effects of old piping, sediments, and biofilms were not investigated in the above-mentioned testing.

Since it is not possible to maintain a distribution system at the same level of cleanliness as a laboratory sample bottle, MPWD was advised to maintain water ages of less than 30 days. Therefore, 16 days was the suggested water age goal for this study. As was explained above in the distribution system operation section, however, meeting this goal could be challenging at times. Maintaining a clean water distribution system is a critical factor in maintaining the high level of delivered water quality and avoiding nitrification events. Distribution system maintenance consists of regular water main flushing and reservoir cleaning. Both of these are discussed in detail below.

REGULATORY CONSIDERATIONS. It is important to note first, however, the regulatory concerns involved with flushing water mains and reservoirs of a disinfected system. Whether the water is chlorinated or chloraminated, the disinfectant residual will need to be removed before the water enters the storm drain. This is best performed by metering a controlled amount of sodium bisulfite to a confined portion of the flow stream. Assuming that a 25% sodium bisulfite solution is used, and that the water has a disinfectant residual of 1 mg/L, approximately 6 mg/L of solution should be metered for dechlorination.

Water main flushing

Developing a program for routine flushing can help reduce the possibility of water quality deterioration. Generally, flushing programs have been established as corrective measures in response to customer complaints or to expel contaminants inadvertently introduced into the system due to backflow problems or line breaks. The advantages to annual flushing are:

- Removal of accumulated sediment
- Removal of older water
- Restoration of residuals to areas of low flow
- Providing an opportunity to alter hydraulic configuration of the system

There are two popular approaches to flushing: unidirectional and conventional. Each approach has a specific use and can help to meet specific water quality goals. Each approach can be implemented on a comprehensive, system-wide basis or on a narrower “spot” basis. Moreover, each approach has a specific use and can help to meet specific water quality goals. This largely depends on the configuration of the system within the area of interest and the water quality goals for that particular area. Comprehensive flushing addresses the entire water distribution system. Spot flushing is typically used to target an acute problem in an isolated area of the system, or in an area where chronic water quality problems (due to low demand or dead-end mains) have led to repeated customer complaints.

Spot flushing is typically implemented at a higher frequency than comprehensive flushing. Since spot flushing focuses more on replacing the bulk water (versus cleaning the pipe), water quality benefits are typically short-lived. However, in the absence of any flushing program, it is likely still beneficial to conduct spot flushing at dead ends. Flushing an area with low demand on a monthly or quarterly basis, for example, would be considered spot flushing.

UNIDIRECTIONAL FLUSHING. Unidirectional flushing consists of isolating a particular pipe section or loop and exercising the hydrants in an organized, sequential manner. This is done by moving from the turnouts (cleanest water) and out into the system, always moving older water out ahead of clean water. This practice can help enhance water quality by cleaning the pipe and restoring a disinfectant residual to that area of the system. This practice also aids in distribution system maintenance by exercising the isolation valves and identifying those that need repair.

Flushing velocities may vary according to the water quality objective in that area of pipe. Moving silts and sediment requires velocities equal to or greater than 3 feet per second. To promote scouring, velocities equal to or greater than 5 feet per second are recommended.

CONVENTIONAL FLUSHING. Conventional flushing consists of opening hydrants in a specific area of the distribution system until preselected water quality criteria are met. These criteria could include detectable disinfectant residual, reduction/elimination of color, reduction in turbidity, etc. It is important to note that valve isolation is not part of conventional flushing. By using valve isolation, as with unidirectional flushing, water is forced through a single main at high velocities capable of dislodging biofilms or other deposits. During conventional flushing, the water used to flush a main may or may not originate from a previously flushed segment. The cleaning efficiency is not as high as with unidirectional flushing. Yet, it is possible to use conventional flushing to restore disinfectant residual or expel water of poor quality from a specific area of the system.

Reservoir cleaning

Prior to chloramine conversion, cleaning the storage reservoirs to eliminate sediment helps reduce the potential for nitrification. Sediment can provide nutrients to microorganisms and bacteria that take part in the nitrification process. In addition, when sediment is suspended in the tank during filling, it can shield bacteria from the disinfectant. In general, reservoirs should be cleaned every three to five years after the initial cleaning. Otherwise, the reservoirs should be inspected annually to assess the rate and level of sediment buildup and to develop a particular cleaning schedule.

Water quality monitoring

Monitoring before, during, and after the conversion to chloramines is an important tool to identify and anticipate the potential for nitrification in the distribution system. Predominately, nitrification is suspected in chloraminated water systems when chloramine residuals are depleted and heterotrophs increase rapidly in an area of the distribution system (Kirmeyer et al., 1995). Once this has happened, nitrifying bacteria may have already become established, and any further control may be difficult to implement successfully. Table 3 presents a general nitrification monitoring program that was developed by Kirmeyer et al. (1995).

Experience has shown that the best way of determining if and to what degree nitrification is occurring is through developing a chemical balance for nitrogen species (Kirmeyer et al., 1995). It was recommended that total ammonia, free ammonia, nitrite, and nitrate are measured at the turnouts and then at the tanks and several locations throughout the distribution system. A good indicator of nitrification is if, at a reservoir, there is a noted decrease in ammonia and increase in nitrite and nitrate from what was measured at the turnouts.

Other suggested monitoring parameters included chloramine residual, temperature, pH, dissolved oxygen, and HPC (R2A agar). HPC can be used as an early warning indicator of nitrification. While standard plate counts (SPC) are helpful, experience has shown they are not as sensitive to changes as counts using the R2A agar (Kirmeyer et al., 1995). Dissolved oxygen has also been successfully used by some utilities to detect nitrification.

Ideally, water quality monitoring data from the turnouts would be collected in a coordinated program along with distribution system water quality data. Turnout water quality data are important because they form baseline information about ammonia, nitrite, nitrate, total organic carbon, temperature, and seasonal water quality trends. Such information may be important to developing a nitrification control strategy. In MPWD's case, the pretreated water may vary somewhat, making it important to coordinate with the SFPUC regarding any changes to the delivered water quality.

Table 3 General nitrification monitoring program

Constituent	SFPUC Water From Turnouts	Reservoirs (depending upon size)	Dead-end Mains	Coliform Stations
Free ammonia	1	1	1	1
Total ammonia	1	1	1	1
Nitrite	1	1	1	1
Nitrate	1	1	1	1
Chlorine dose	1			
Total chlorine residual	1	1	1	1
Inorganic chloramines	2	2		
Organic chloramines	2	2		
Temperature	3	2	2	3
pH	4	3	4	4
HPC (R2A Agar)	1	1	1	1
Alkalinity	4	3	4	4
Dissolved oxygen	4	2	2	3
TOC/DOC/AOC	3	3		

1) Primary monitoring parameters B are strongly recommended.
2) Secondary monitoring parameters B are strongly recommended for all areas in the MPWD system that experience any low residual levels or other signs of nitrification.
3) Recommended for problem areas.
4) Supplement with discretionary monitoring.

Source: Kirmeyer et al. (1995)

An excellent method of identifying potential nitrification problems in reservoirs is to collect samples from various depths and various locations throughout the reservoir. These samples could be compared to detect evidence of stagnation areas. The most important water quality parameter in this type of sampling is the free chlorine or chloramine residual, because in parts of the reservoir where circulation is poor, the disinfectant residual will likely be lowest, or possibly absent. Likewise, where circulation and mixing are good, the residual should be higher. This monitoring approach is highly recommended for systems converting to chloramines and should be implemented prior to conversion. By monitoring free chlorine residual in reservoirs prior to the startup of chloramination, operators will already know which reservoirs may be prone to nitrification.

In addition to maintaining a database of routine flushing periods and results, water quality monitoring should be conducted at dead-end locations. The water quality parameters are the same as those for monitoring reservoirs. In addition, monitoring at coliform stations is important because there is already a historical database and the utility already visits and collects samples at these sites. These locations form a fingerprint for the entire distribution and provide an early warning network.

Summary of distribution maintenance recommendations

Based on the above, the following recommendations to MPWD were made for distribution system cleaning and water quality monitoring:

- *Develop and implement a comprehensive flushing program*. Routine unidirectional, system-wide flushing should be conducted before the initiation of chloramination and continued afterward. This will help reduce chloramine degradation in the reservoirs and far reaches of the distribution system.
- *Clean the reservoirs before conversion*. Performing reservoir cleaning in advance of the conversion to chloramines makes sense because sediment in a reservoir can provide nutrients to microorganisms involved in nitrification. Starting with clean reservoirs reduces the potential for nitrification and allows operators to eliminate this factor later on should a potential nitrification event occur.
- *Begin an expanded water quality monitoring program one year before conversion*. A system-wide monitoring program should begin a full year ahead of the conversion to chloramine and continue after the conversion. Table 3 presents appropriate monitoring parameters and general locations. Establishing a database with at least one annual cycle of monitoring results prior to conversion will allow the operators to understand seasonal effects so that they are not misinterpreted as being caused by the conversion.

- *Develop and implement a detailed reservoir monitoring program.* An excellent method of identifying potential nitrification problems in reservoirs is to collect samples from various depths and various locations throughout each reservoir. These samples can be compared to detect evidence of stagnation. The most important water quality parameter in this type of sampling is the free chlorine (after conversion, chloramine) residual, because in parts of the reservoir where circulation is poor, the disinfectant residual will likely be lowest, or possibly absent. Likewise, where circulation and mixing are good, the residual should be higher.
- *Respond to problems during full chloramine operational status.* Having implemented the above recommendations, there is a possibility that the conversion to chloramines will cause no unexpected or immediate problems. The water quality database with at least one annual cycle of monitoring results will provide the frame of reference needed to manage problems as they occur. For example, it should be possible to identify particular reservoirs that might be losing residual, and then to respond appropriately by boosting with disinfectant, by increasing circulation, or by decreasing water age.

SUMMARY

The intent of this study was to provide MPWD with tools for preparing its system for the upcoming conversion to chloramines as well as insights into the effects that chloramines could have on the system. Chloramines will likely affect three main areas within the system:

- *System materials.* As a result of this study, corrosion of the distribution system piping is not likely to increase in the presence of chloramines. However, elastomeric components in the system may experience some increased degradation, but the rate of this depends on the age and quality of the material as well as the water quality in the system. A recommendation was made that MPWD begin a public education outreach program to explain the possible effects of chloramines to its customers. In addition, the operations staff of MPWD should monitor the unaccounted-for water in the system and be alert to any increases possibly signaling leaking materials.

- *System design and operation.* This study analyzed the water age within the MPWD system as a method of determining the potential for nitrification. During minimum day conditions (low demand), water will need to be moved more quickly through the system. Operational schemes were recommended to help control the water age to within 16 days for the majority of the system. In addition, modifications to the reservoirs were suggested to reduce the stagnant areas within them. Finally, an annual flushing program was recommended to reduce stagnant water in dead-end mains, which would help to preserve the water quality in low-demand areas of the system.
- *System maintenance.* Two recommendations were made with regard to system maintenance: (1) maintain a clean system that does not harbor bacteria that could cause nitrification and (2) monitor water quality for changes that might signal nitrification. System flushing, either comprehensive or unidirectional, can aid in controlling bacteria population in the system. Reservoir cleaning prior to conversion to chloramines will also ensure that the reservoir walls are not harboring bacteria. Finally, water quality monitoring both before and after conversion was suggested as a way to identify potential problem areas in the distribution system.

The recommendations of this study were made specifically for the MPWD's water system. However, many of the same concepts apply to any water system considering conversion from chlorine disinfection to chloramines. Finally, as with any significant water treatment change, utilities planning to convert to chloramines are encouraged to begin a public outreach program long before the conversion. This program should educate customers on the reasons for the conversion as well as the risks associated with it. Keeping the customers informed about the diligent efforts the utility is making in preserving water quality and regulatory compliance can greatly enhance the image of the utility as well as its ability to implement necessary improvements.

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CHAPTER · 15

Success! Switching From Chloramines to Chlorine in the Distribution System: Results From a One-Year Full-Scale Trial

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INTRODUCTION

Approximately 10 years ago, modifications were made to the Oakville Water Purification Plant (WPP) to allow the use of chloramines for residual maintenance. Since then the drivers for secondary disinfection in this system have changed. Progress is being made with the ongoing efforts of the Accelerated Replacement Program for the Cast Iron Watermains in the South Halton System (Cast Iron Main Replacement Program), the benefits of which include improved disinfectant residual persistence and a decrease in colored water incidents in historical problem areas.

In an effort to manage growth, the Oakville system (a 109 ML/d conventional plant) was merged with the Burlington system (a 218 ML/d direct filtration plant) in the fall of 1999. Both systems are owned and operated by the Regional Municipality of Halton (the Region). Of concern were the potential impacts associated with blending: although both plants treat surface water from Lake Ontario, free chlorine is used for residual maintenance in the Burlington system. After a detailed cost-benefit analysis, the water supplier opted to examine the feasibility of using free chlorine for residual maintenance in both systems, rather than operating the joined systems with a blend of chlorinated and chloraminated water.

Following the recommendation to proceed to a full-scale trial with free chlorine in the Oakville system, implementation and contingency plans were prepared. Evaluation criteria and a public notification strategy were developed as part of the implementation and contingency plans.

The one-year trial was initiated on October 18, 1999, and continued to October 31, 2000. During this period, regulations affecting drinking water supplies in Ontario were changed. This did not affect the goals and objectives of the one-year full-scale trial or the evaluation criteria established for the trial. However, the introduction of the drinking water regulation affected how the systems were operated and how the data were reviewed. A distinction was made for data collected before August 26, 2000 (and therefore subject to the 1994 Ontario Drinking Water Objectives [ODWO]) and for data collected after August 26, 2000 (and subject to the 2000 Ontario Drinking Water Standards).

The results from the one-year full-scale trial are presented herein. The recommendation to proceed with the one-year full-scale trial was made to the Region in June 1999. Implementation and contingency plans, including the identification of evaluation criteria and the development of a public notification strategy, were prepared during the summer of 1999. The full-scale trial commenced on October 18, 1999, and continued to October 31, 2000.

OBJECTIVES

On October 18, 1999, the ammonia feed system at the Oakville WPP was turned off, and water carrying a free chlorine residual was distributed to the Oakville system.

The goal of the one-year full-scale trial with free chlorine in the Oakville system was to assess the performance of the Oakville system when using free chlorine for residual maintenance. To meet this goal, the objectives of the one-year full-scale trial were to:

1. Demonstrate that the water in both the Oakville and Burlington systems is of similar quality.
2. Demonstrate the persistence of a free chlorine residual in the Oakville system and perform a direct comparison of the Oakville system with the Burlington system, where both systems are using free chlorine.
3. Demonstrate that the water quality in the Oakville system has been maintained or improved, relative to operating the system with chloramines.

REGULATORY ISSUES

In August 2000, the Ontario Ministry of Environment (MOE) introduced Regulation 459/00 under the Ontario Water Resources Act. Prior to this, the quality of drinking water was subject to the guidelines of the Ontario Drinking Water Objectives (1994), a document that did not have the legal force of law. As of August 26, 2000, drinking water supplies are legally required to meet the criteria of the Ontario Drinking Water Standards (ODWS, August 2000, included as part of Regulation 459/00). The ODWS differ from the ODWO with respect to disinfection practices, namely:

- Requirements for primary disinfection, including a minimum 3-log removal/inactivation of *Giardia* cysts and a 4-log removal/inactivation of viruses.
- Requirements for secondary disinfection, including the presence of a disinfectant residual throughout the distribution system at a concentration ≥ 0.2 mg/L free chlorine (or ≥ 1.0 mg/L as combined chlorine, if appropriate).
- The ODWO required disinfectant residuals greater than detection in ≥ 95% of monthly samples; this was not addressed in the ODWS.
- Limits on the use of disinfectants in the distribution system (4.0 mg/L free chlorine or 3.0 mg/L combined chlorine at all times at any location in the system).

Definitions for “unsafe” and “deteriorating” water quality under the ODWO have been replaced with “indicators of adverse water quality” in the ODWS, where the latter include the distribution of unchlorinated water (defined as distribution system samples with < 0.05 mg/L free chlorine or < 0.25 mg/L combined chlorine for chloraminating systems).

The Region has a procedure in place to respond to conditions of potentially unsafe drinking water. The response taken by the Region depends on the nature of the bacteriological test results (quoted from the Region’s Unsafe Drinking Water Response Procedure, spring 2000):

Incident	Definition
Level 1	A first single sample report demonstrating the presence of total coliform in a supply (TC > 0 cfu/100 mL; P/A = fail 24 hr or 48 hr) or an exceedance of the ODWO for either heterotrophic plate count (HPC > 500 cfu/mL) or total coliform background (TCB > 200 cfu/100 mL)
Ongoing Level 1	A resample that continues to exceed the ODWO for TC, HPC, or TCB; however, (shows) improvement due to remedial action
Level 2	An exceedance of any other health-related ODWO where there is no evidence of improvement in the analysis results, including *E. coli*, or a repeat (or multiple) unsatisfactory sample result for TC, HPC, or TCB

In general, the Medical Officer of Health determines the status of an incident. An Ongoing Level 1 Incident may continue for prolonged periods of time provided that no *E. coli* is measured during the incident. The Region has updated its Unsafe Drinking Water Response Procedure to address the requirements of Regulation 459/00 (approved in March 2001).

METHODS

Public Notification

During October 1999, the Region notified customers using the following methods:

- Ten days before the trial started, a pamphlet was made available to the general public and mailed to special users, including hospitals and doctors, as well as pet stores, pools, and pool operators.
- Information was posted to the Region's Web site. A media release was prepared for local newspapers, and a notice of change was placed in local papers on the weekend before the trial started.
- A telephone hotline was established.

Analytical Methods

Analytical methods used during the one-year full-scale trial are summarized in Table 1. Unless otherwise noted, Region staff conducted all analytical work.

Table 1 Summary of analytical methods

Parameter	Description	Reference
Free and total chlorine (field)	DPD colorimetric method using an HACH pocket chlorimeter	Modified Standard Methods 4500-Cl G
Chlorine speciation (mono, di/tri, total at lab)	Amperometric titration using a Wallace & Tiernan titrator	Standard Methods 4500-Cl
Presence/absence	Selective microbiological test performed to obtain qualitative information on coliforms	Presumptive presence/ absence test based on Standard Methods 9215D
Total plate count	Membrane filtration	Based on Standard Methods 9215D
Total coliform background	Membrane filtration	Based on Standard Methods 9222B, 9222D, and DC medium by MOE/MOH
Total coliform	Membrane filtration	Based on Standard Methods 9222B, 9222D, and DC medium by MOE/MOH
E. coli	Membrane filtration	Based on Standard Methods 9222B, 9222D, and DC medium by MOE/MOH
Free ammonia		HACH method based on and adapted from CLIN.CHIM.ACTA.14403 (1966)
Nitrite and nitrate	Colorimetric	Standard Methods 4500-NO2 and 4500-NO3
Trihalomethanes (outside lab)	Purge and trap, GC/MS	USEPA Method 624

Implementation Plan

An implementation plan was prepared to guide the conversion from chloramines to free chlorine in the Oakville system. Of concern was the potential occurrence of temporary taste and odor problems as the blending interface traveled across the system. Potential problem areas included reservoirs, dead ends of trunk mains, and some isolated residential sections.

No changes were made to the operations of the Burlington system during the one-year full-scale trial.

Operations during implementation

On October 18, 1999, the ammonia feed system at the Oakville WPP was turned off to allow the distribution system to be operated with free chlorine for residual maintenance. The system was not flushed before the conversion. Prior to system turnover, reservoir levels were lowered to reduce the detention time of the system as a whole and limit the potential problems associated with blending chlorinated and chloraminated waters. Sodium hypochlorite was manually added to select reservoirs to encourage the presence of a free chlorine residual. Localized flushing was carried out at the western end of Zone 1 (pressure zone closest to the treatment plant) to restore residuals in that area.

Temporary booster chlorination facilities

The need for temporary booster chlorination facilities at the various facilities in the Oakville system was identified in the implementation plan. Temporary facilities were employed during system conversion (as per Table 2) and maintained for the duration of the trial.

Target residuals

To convert the chloraminated water present in the distribution system to free chlorine, the system was operated beyond the chlorine breakpoint to ensure that a free chlorine residual was established. This potentially required an increase of more than 50% in the total chlorine residual leaving the plant (to approximately 1.4 mg/L) and a chlorine-to-ammonia ratio near 10:1 (as Cl_2:NH_3-N). Efforts were made to avoid operating near the breakpoint (7.6:1 Cl_2:NH_3-N) to reduce the potential for loss in chlorine residual that may occur.

Once it could be established that ammonia levels in the distribution system returned to baseline conditions (i.e., ammonia levels in the

Table 2 Preliminary assessment of the logistics of adding chlorine in the Oakville system during system conversion

Facility	Influent	Effluent
McCraney Reservoir	Add NaOCl via the access hatch on the inlet, use a metering pump	
Eighth Line Reservoir	Add NaOCl to the header as the reservoir is filling	Add NaOCl to the header as the reservoir is emptying
Moore Reservoir*†	Add NaOCl via the access hatch on the inlet; use a metering pump	
Upper Middle Road Reservoir*	Permanent NaOCl metering system installed	Permanent NaOCl metering system installed
Eighth Line Pumping Station*	Temporary NaOCl metering system installed	Temporary NaOCl metering system installed

* Maintained for the duration of the one-year full-scale trial.

† Permanent facilities were installed in 2001.

distribution system tracked with raw water levels of about 0.05 mg/L), the target residual in plant effluent was reduced to ≈0.8 mg/L free chlorine to match the Burlington system.

Targets for chlorine residuals in treated water leaving the Oakville and Burlington WPPs and at booster chlorination facilities are identified in Table 3. Note that targets were increased in September and October to meet the requirements for residual maintenance per the ODWS.

Enhanced distribution system monitoring plan

A key component of the one-year full-scale trial was the development of an enhanced monitoring program. The monitoring program addressed three sampling periods:

1. An initial 48-hour period immediately after the ammonia was turned off, during which time residuals in the reservoirs and system were monitored intensively.
2. The first two weeks following conversion, during which time residuals were monitored throughout the system to identify

Table 3 Summary of targets for chlorine residuals in treated water and booster chlorination facilities

System	Location	Date	Target
Oakville	Oakville WPP	Before September 21, 2000	0.8 to 1.0 mg/L free chlorine
		After September 21, 2000	1.2 mg/L free chlorine
	Upper Middle Road Reservoir, outlet		1.0 mg/L free chlorine
	Eighth Line Booster Station		1.0 mg/L free chlorine
Burlington	Burlington WPP	Before August 24, 2000	0.7 mg/L free chlorine
		After August 24, 2000	1.0 mg/L free chlorine
	Burlington Reservoirs		1.0 to 1.2 mg/L free chlorine

areas where maintaining a residual was difficult (less than 0.2 mg/L as free chlorine) and areas with potential nitrification problems.

3. Ongoing monitoring continued for the remainder of the one-year full-scale trial, the purpose of which was to document residual persistence and bacteriological quality throughout the system and to identify potential problem areas.

The locations included in the existing distribution system monitoring program were modified to ensure adequate coverage of the system, including areas with small-diameter pipe. Twenty-four locations were identified in the Oakville system for residual monitoring during the initial 48-hour implementation period and the first two weeks of monitoring, of which 12 of the 24 locations were new. Parameters monitored during the ongoing monitoring period are listed in Table 4, including the frequency with which each parameter was monitored. Sampling efforts in the Burlington system were not modified.

During the third month of the trial, the number of sample locations was reduced to pretrial levels. In addition to the 12 traditional monitoring sites, 5 enhanced sampling program locations were retained due to

Table 4 Recommended ongoing monitoring program in the Oakville system

Parameter	Frequency	Number of Sample Locations	Comment
Free and total chlorine	Weekly	Existing monitoring program	All 4 reservoirs (influent and effluent)
Presence/absence, total plate count	Weekly	Existing monitoring program	Increase frequency and sample locations in areas where it is difficult to maintain a residual
Total coliform background, total coliform, *E. coli*	Weekly as needed	Existing monitoring program	No change to existing monitoring efforts
Trihalomethanes	Monthly	Plant effluent ≥ four locations in the distribution system	
Alkalinity, hardness, conductivity, dissolved organic carbon	Quarterly	Existing monitoring program	No change to existing monitoring efforts
Ammonia, nitrite, nitrate, and nitrogen	Quarterly	Existing monitoring program	No change to existing monitoring efforts

initial results with relatively higher bacteriological parameters and/or trihalomethanes and relatively lower disinfectant residuals.

Sampling efforts in the Burlington system were not modified during the one-year full-scale trial.

Evaluation Criteria

Criteria for evaluation are listed in Table 5; relevant regulatory objectives (ODWO, 1994) and standards (ODWS, 2000) have been included to provide a comparison.

For the purpose of this study, the running annual average was used, based on the quarterly average of monthly average historical levels across the system. As per the ODWS, compliance is based on the most remote location in the distribution system, calculated as the running annual average using the sample collected after August 26, 2000.

Results for the Oakville system using free chlorine (October 18, 1999, through October 31, 2000) were compared with results for the Burlington system using free chlorine for the same time period. Additionally, historical data based on chloramine use in the Oakville system in a four-year period before the full-scale trial were compared with data based on chlorine use in the Oakville system, as collected during the full-scale trial.

Data were evaluated on a monthly basis (using all samples collected throughout each system). Monthly data were reviewed for each location in the Oakville and Burlington systems to identify areas where it was difficult to maintain a residual or where conditions exceeded the targets of Table 6. It is noted that historical trihalomethane data from 1996 through 1999 were reviewed to supplement data collected during the one-year trial.

The ODWS came into effect during the last two months of the full-scale trial. The Region's treatment works were operated to comply with the ODWO until August 26, 2000. As of August 26, the treatment works were operated to meet the requirements of the ODWS, most noticeably to achieve a residual greater than 0.05 mg/L free chlorine, and preferably in excess of 0.2 mg/L free chlorine, at all sites throughout the distribution system. Similarly, data for all other parameters were reviewed with respect to the guidelines or regulations in effect at the time the sample was collected.

Table 5 Evaluation criteria for one-year full-scale trial

Parameter	Criterion	Basis for Evaluation	ODWO 1994	ODWS 2000
Residual persistence	Free chlorine ≥ 0.2 mg/L across the system	Frequency of occurrence on a monthly basis, whole system	Detected in 95 percent of monthly samples (> 0.03 mg/L)	> 0.05 mg/L free chlorine
Presence/absence	No detects	Frequency of occurrence on a monthly basis, whole system	No detects	No detects
Total plate count	≤ 500/mL	Frequency of occurrence > 500/mL on a monthly basis, whole system	≤ 500/mL	≤ 500/mL
Total coliform background	≤ 200/100 mL	Frequency of occurrence > 200/100 mL on a monthly basis, whole system	≤ 200/100 mL	≤ 200/100 mL
Total coliform	No detects	Frequency of occurrence > 0/100 mL on a monthly basis, system-wide	No detects	No detects
E. coli	No detects	Frequency of occurrence > 0/100 mL on a monthly basis, system-wide	No detects	No detects
Customer complaints		Complaints received on a monthly basis	n/a	n/a
Trihalomethanes	≤ 100 µg/L	Running annual average	100 µg/L	100 µg/L

RESULTS AND DISCUSSION

All data were reviewed to determine whether or not the water quality in the Oakville system was maintained, degraded, or enhanced as a result of the conversion to free chlorine for residual maintenance. Data were tabulated with respect to the occurrence of deteriorating or unsafe water, based on the evaluation criteria of Table 5. This analysis is based on the frequency with which parameters exceed the criteria of Table 5, in light of the objectives or standards in effect at the time the sample was collected.

During the full-scale trial, an *E. coli* outbreak occurred in Walkerton, Ontario, in May 1999. The extensive media coverage surrounding this event included discussions on the safety of drinking water supplies, one by-product of which may be a greater awareness of water quality issues in the province as a whole. What impact this had on the customer feedback received during the trial cannot be determined.

Bacteriological Quality and Residual Persistence

Water quality in the Oakville system: regulatory compliance

Bacteriological data collected during the full-scale trial were reviewed for conditions of adverse, deteriorating, and unsafe water. During the study period, at no time was the water in the Oakville system considered unsafe, as per the definition of the objectives or standards in place (see Table 6). Positive results were not recorded for *E. coli* and total coliform (TC), and results for total coliform background (TCB) and total plate count (TPC) were lower than the criteria of Table 5 (≤ 200 cfu/100 mL and ≤ 500 cfu/mL, respectively). During the period from October 18, 1999, to August 26, 2000, nine occurrences of deteriorating water quality were reported, eight of which were due to a single positive result for the presence/absence (P/A) test. The remaining occurrence was due to the presence of TPC in excess of 500 cfu/mL. In all cases, resampling was conducted and the safety of the water supply was confirmed. After August 26, 2000, one positive result from the P/A test was reported, suggesting an indicator of adverse water quality; again, resampling confirmed the safety of the water.

Data for chlorine residuals were reviewed for regulatory compliance. Residual persistence is one indicator of performance, and observations of decreasing chlorine residuals should be used as a flag that there might be a problem. When residuals are lower than an action level, other bacteriological data should be reviewed. By knowing how the system behaves throughout the year, the appropriate action can be implemented.

Table 6 Bacteriological quality in the Oakville distribution system[*] (October 18, 1999, to October 31, 2000)

Parameter	Number of Samples Collected	Number of Samples Safe	Number of Samples Indicating Adverse Water Quality[†]	Number of Samples Unsafe[‡]	Number of Samples Deteriorating[‡]
E. coli	59	59	0	0	n/a
Total coliform	59	59	0	0	0
Total coliform background	34	34	0	n/a	0
Total plate count	1,958	1,957	0	n/a	1[§]
Presence/absence	1,933	1,924	1[§]	0	8[§]

[*] Data for treated water from the plant were removed from this analysis. Samples were reviewed with respect to the objectives or standards in place at the time of sample collection.

[†] Applied to samples collected after August 26, 2000 (and subject to the ODWS, 2000).

[‡] Applied to samples collected before August 26, 2000 (and subject to the ODWO, 1994).

[§] Resampling confirmed the safety of the water supply.

Between October 18, 2000, and August 26, 2000, free chlorine residuals were not detected at sample locations in the Oakville distribution system on three occasions. This complied with the ODWO condition that a residual be detected in at least 95% of monthly samples. The bacteriological quality of the water was deemed safe, and the disinfectant residual was restored in subsequent samples collected at the same location.

With the introduction of the ODWS on August 26, 2000, data for disinfectant residuals were reviewed for instances when the residual measured was less than 0.2 mg/L free chlorine and less than 0.05 mg/L free chlorine. The presence of a residual in excess of 0.2 mg/L free chlorine can be viewed as a target, and MOE notification is not necessary should a residual less than 0.2 mg/L free chlorine be detected. However, results with less than 0.05 mg/L free chlorine do require MOE notification. Residuals less than 0.2 mg/L free chlorine were measured in approximately 20% of Oakville system samples collected in September 2000. The target chlorine residual in water discharged by the Oakville WPP was raised on September 21, 2000, and as a result, residuals in excess of 0.2 mg/L free chlorine were measured during the last month of the full-scale trial. On one occasion, a result less than 0.05 mg/L free chlorine was measured (September 19, 2000, after which chlorine residuals were increased); all bacteriological indicators complied with the criteria of Table 5, and a residual in excess of 0.2 mg/L free chlorine was measured in the subsequent sample.

Results addressing residual persistence in the Oakville system are summarized in Table 7.

Water quality in the Burlington system: regulatory compliance

Bacteriological results for samples collected in the Burlington system were reviewed for conditions of adverse, deteriorating, and unsafe water. From October 18, 1999, to August 26, 2000, there were no occurrences of unsafe water. During this same period (i.e., subject to the ODWO of 1994), there were nine occurrences of deteriorating water quality: four were due to the presence of TC, two were due to TPC > 500 cfu/mL, and three were due to a positive P/A test. Resampling associated with these events confirmed the safety of the water supply and that residual of ≥ 0.2 mg/L free chlorine was present. Between August 26, 2000, and the end of the study period, there were no indicators of adverse water quality, as defined by the ODWS (2000).

Data for chlorine residuals were reviewed for regulatory compliance. Prior to August 26, 2000, free chlorine residuals were not detected at

Table 7 Residual persistence in the Oakville and Burlington distribution systems[*] (October 18, 1999, to October 31, 2000)

	Number of Samples Collected	Number of Samples Pass	Number of Samples < Operating Guideline (ODWS, 2000) < 0.2 mg/L Free Cl_2	Number of Samples Indicating Adverse Water Quality (ODWS, 2000) < 0.05 mg/L Free Cl_2	Number of Samples Non-Detect (ODWO, 1994) < 0.03 mg/L Free Cl_2
Oakville					
Before Aug. 26, 2000	1,553	1,550	n/a	n/a	3
After Aug. 26, 2000	301	300	29	1[*]	n/a
Burlington					
Before Aug. 26, 2000	1,159	1,158	n/a	n/a	1
After Aug. 26, 2000	266	266	26	0	n/a

[*] The chlorine residual was increased and residuals > 0.2 mg/L free chlorine were measured in subsequent samples.

sample locations in the Burlington distribution system on one occasion. This complied with the ODWO condition that a residual be detected in at least 95% of monthly samples. The bacteriological quality of the water in samples collected simultaneously was deemed safe, and the disinfectant residual was restored in subsequent samples collected from the same location.

Data collected after August 26, 2000, were reviewed for instances when the residual measured was less than 0.2 mg/L free chlorine and less than 0.05 mg/L free chlorine.

Residuals less than 0.2 mg/L free chlorine were measured in approximately 40% of Burlington system samples collected in the remainder of August 2000. The target chlorine residual in water discharged by the Burlington WPP was raised on August 24, 2000, and as a result, residuals in excess of 0.2 mg/L free chlorine were measured in more than 85% of samples collected during September 2000. The average chlorine residual in water leaving the Burlington WPP was steadily raised during the fall of 2000. During the last month of the full-scale trial, a residual in excess of 0.2 mg/L free chlorine was measured in all samples collected. Results with less than 0.05 mg/L free chlorine were not observed after August 26, 2000. Results addressing residual persistence in the Burlington system are summarized in Table 7.

Summary of water quality in the Oakville and Burlington systems with free chlorine

Although during the winter it is more difficult to comply with disinfection requirements (based on the *CT* concept for *Giardia* and viruses) at the Burlington WPP than at the Oakville WPP, both facilities produced water of a comparable quality. Efforts to improve the disinfection barriers at both plants are under way: the addition of ozone is currently under design for the Burlington WPP, and ozone is being considered at the Oakville WPP. However, the Burlington system performs comparably with the Oakville system in terms of water quality performance in the distribution system. In Oakville, the ongoing efforts of the Accelerated Replacement Program for the Cast Iron Watermains in the South Halton System (Cast Iron Main Replacement Program) may be providing water quality benefits in addition to reducing the occurrence of red water. In Burlington, microbes could potentially hide in tubercles and aluminum-based material deposits present at the pipe wall. These bugs can then slough off or be released into the bulk water by changes in water quality and hydraulics. Process modifications at the Burlington WPP to address material deposits found in the distribution system are under design.

Based on the occurrence of conditions defined as deteriorating or unsafe and indicators of adverse water quality, the Oakville system performs comparably to the Burlington system (see Table 8), suggesting the successful use of free chlorine in the Oakville system with respect to bacteriological quality and residual maintenance.

Water quality in the Oakville system with free chlorine and chloramines

The historical performance of the Oakville system with chloramines was compared with the one-year full-scale trial with free chlorine, the results of which are presented in Table 9. Data used in Table 9 are based on summary tables of performance included in the Region's Annual Reports for the Oakville system (1996 to 1999).

The water quality performance of the Oakville system was maintained or improved upon when operating the system with free chlorine with respect to controlling *E. coli* (maintained), TC (enhanced), TCB (maintained), TPC (enhanced), and P/A.

Customer Complaints

The complaints received each month from January 1996 through October 2000 were categorized by type of complaint and totaled on a monthly basis. Historical data were included in the review to identify trends.

Since operating the Oakville system with free chlorine (October 18, 2000), there has been a noticeable increase in chlorinous T&O complaints in Oakville (see Figure 1). This was most pronounced during the system turnover (when a total of 13 complaints were received), but complaints were received regularly throughout the year 2000. Although not unexpected, more complaints of chlorine were received when operating the system with free chlorine (2000) compared with the use of chloramines (1996–1999). It did not appear that the customers became accustomed to the chlorinous taste and odor with time, as complaints were received regularly throughout the trial period. Complaints of chlorinous taste and odor increased again in the fall of 2000, when water with higher chlorine residuals was discharged to the system. A similar increase in residuals was made at the Burlington WPP; the impact that this had on customer complaints was less discernible (see Figure 2). It is not possible to determine whether or not the media attention surrounding the *E. coli* outbreak in Walkerton, Ontario, affected the public's perception of the water supply.

There was a general decreasing trend in the number of complaints received with reference to rusty, colored, or dirty water in both systems,

Table 8 Comparison of water quality in the Oakville and Burlington systems (October 18, 1999, to October 31, 2000)

		Oakville		Burlington	
		Bacteriological	Residuals	Bacteriological	Residuals
Before Aug. 26, 2000	Deteriorating[*]	9	3	9	1
	Unsafe[†]	0	–	0	–
After Aug. 26, 2000	Adverse water quality indicator[‡]	1	1	0	0

[*] Single detect TC or P/A, TCB > 200 cfu/100 mL, TPC > 500 cfu/mL, or chlorine residual less than detection (0.03 mg/L free chlorine); this is defined as a Level 1 Incident (ODWO, 1994).

[†] Single detect *E. coli*, multiple detect TC or P/A. If after corrective action there is no bacteriological improvement, this is redefined as a Level 2 Incident (ODWO, 1994).

[‡] Single detect *E. coli*, total coliform, or presence/absence, total coliform background > 200 cfu/100 mL, total plate count > 500 cfu/mL, or chlorine residual < 0.05 mg/L free chlorine (ODWS, 2000).

Table 9 Bacteriological quality in the Oakville distribution system

Year	Parameter	Incidents Indicating Adverse Water Quality (ODWS, 2000)	Incidents Unsafe (ODWO, 1994)	Deteriorating Incidents (ODWO)
1996*	*E. coli*	–	0	–
	Total coliform	–	1	4
	Total coliform background	–	–	1
	Total plate count	–	–	5
	Presence/absence	–	0	1
	TOTAL	–	1	11
1997*	*E. coli*	–	0	–
	Total coliform	–	1	0
	Total coliform background	–	–	1
	Total plate count	–	–	5
	Presence/absence	–	0	2
	TOTAL	–	1	8
1998*	*E. coli*	–	0	–
	Total coliform	–	2	2
	Total coliform background	–	–	0
	Total plate count	–	–	14
	Presence/absence	–	0	9
	TOTAL	–	2	25
1999*	*E. coli*	–	0	–
	Total coliform	–	0	0
	Total coliform background	–	–	0
	Total plate count	–	–	4
	Presence/absence	–	0	9
	TOTAL	–	0	13
2000	*E. coli*	0	0	–
	Total coliform	0	0	0

Table continued next page.

Table 9 Bacteriological quality in the Oakville distribution system *(continued)*

Year	Parameter	Incidents Indicating Adverse Water Quality (ODWS, 2000)	Incidents Unsafe (ODWO, 1994)	Deteriorating Incidents (ODWO)
	Total coliform background	0	–	0
	Total plate count	0	–	0
	Presence/absence	1	0	4
	TOTAL	1	0	4

* Based on data contained in the Region's Annual Reports for 1996 to 1999. Samples were reviewed with respect to the objectives or standards in place at the time of sample collection.

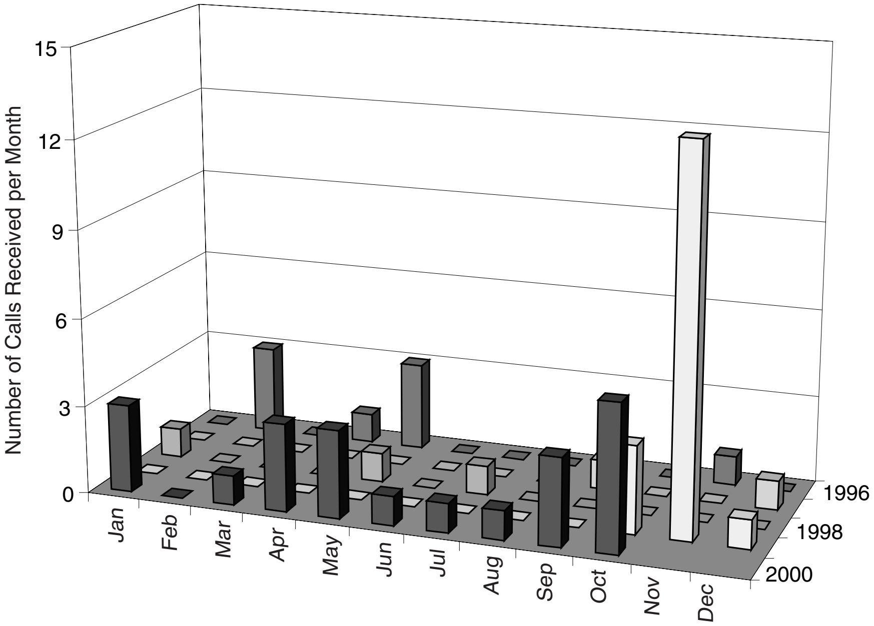

Figure 1 Oakville water system customer calls for chlorinous taste and/or odor

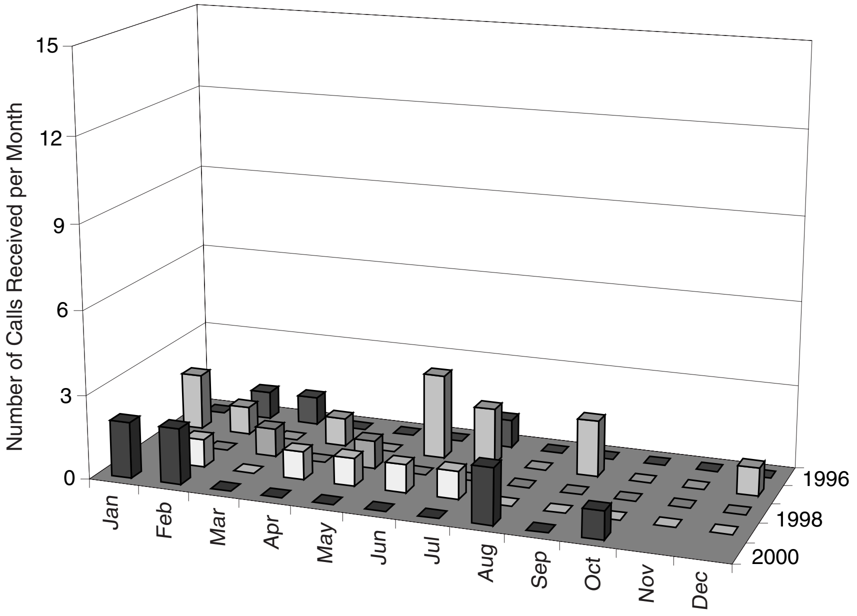

Figure 2 Burlington water system customer calls for chlorinous taste and/or odor

but a greater improvement was observed in the Oakville system (see Figure 3). The conversion from chloramines to free chlorine did not appear to aggravate the occurrence of rusty, colored, or dirty water. It is suggested that the benefits of the Cast Iron Main Replacement Program are being realized with time, particularly in the Oakville system.

Disinfection By-product Formation

The formation of trihalomethanes (THMs) was monitored at multiple locations throughout the Oakville system on a monthly basis from October 1999 through October 2000. Historical data from 1996 to 1999 were used with data collected during the full-scale trial to calculate the running annual average concentration of THMs in each system for the purpose of this study (see Figure 4). Since the start of the full-scale trial (fourth quarter, 1999), THMs in the Oakville system have increased from 11 μg/L to 20 μg/L (based on the running annual average of quarterly monthly averages) but are well below the ODWS of 100 μg/L. This is still

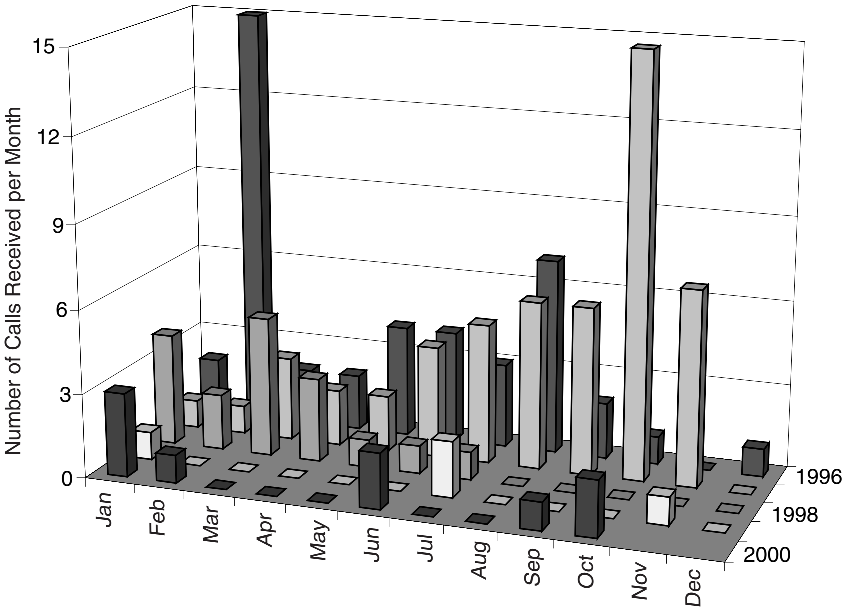

Figure 3 Oakville water system customer calls for rusty, colored, or dirty water

lower than the maximum value calculated for the five years of historical data for the Burlington system, despite operating the Oakville system with relatively higher chlorine residuals.

Identification of Potential Localized Problem Areas

Further evidence of the success of the conversion from chloramines to free chlorine is provided by reviewing historical data for bacteriological quality and residual maintenance from January 1996 through the end of the full-scale trial on the basis of sample location. The purpose of this exercise was to identify localized areas where it was difficult to meet the performance metrics for water quality (residual maintenance and/or the frequency with which various bacteriological indicators were present). In the Oakville system, locations that repeatedly had difficulties are listed in Table 10.

Difficulties with residual persistence occurred at sample locations toward the end of the system and at reservoirs, where water is relatively older. However, residual persistence was improved at select sample

Table 10 Potential problem areas in the Oakville system: A comparison of historical performance with the full-scale trial

	Difficulty with Bacteriological Indicators		Difficulty with Residual Persistence		
Location	1996–1999*	Full-Scale Trial†	1996–1999*	Full-Scale Trial†	Action by Region
Eighth Line Booster Station	No	No	Yes	No	Booster chlorination is working and will be maintained.
Moore Reservoir/Inlet/ Outlet	N.M.‡	No	N.M.	Yes	Permanent booster chlorination works were added as part of the expansion to the Moore Reservoir in 2001. Improvements are expected.
Nottinghill Gate & Pilgrims Way	No	No	Yes	No	Improvements were observed with free chlorine.
South Service Rd Garage	No	No	Yes	No	The Region is monitoring this location; residual persistence improved as the full-scale trial progressed.
System North	No	No	Yes	Improving	Permanent booster chlorination works were added as part of the expansion to the Moore Reservoir in 2001. Improvements are expected.
System North-Northeast	No	No	Yes	Improving	Booster chlorination is working and will be maintained.
System South	Yes	No	Yes	No	Improvements observed with free chlorine.
Zone 4 Booster Station	Yes	Yes	No	No	Booster chlorination facilities may be added upon expansion.

* Based on a review of historical data collected from 1996 to 1999 when operating the system with chloramines

† Based on data collected during the one-year full-scale trial when operating the system with free chlorine

‡ This site was not monitored during the indicated period (i.e., a new sample location added for the full-scale trial).

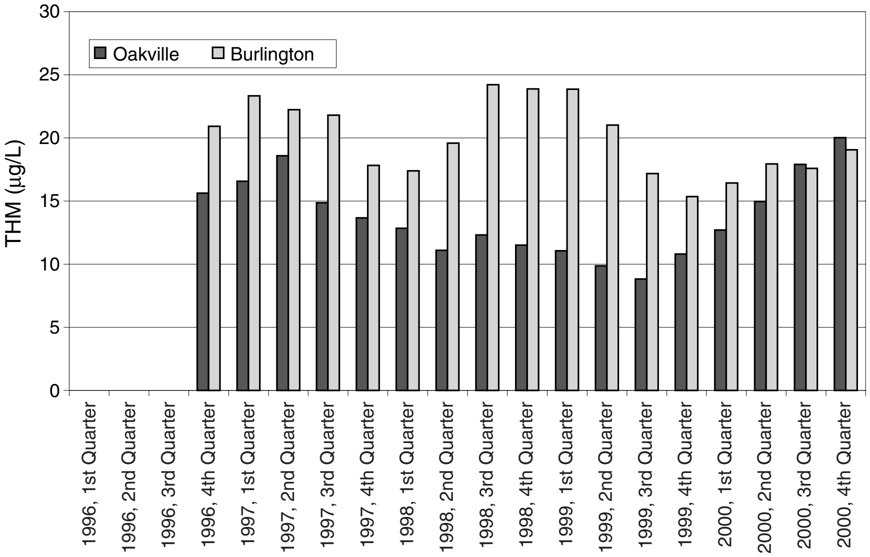

Figure 4 Trihalomethanes in the distribution system, running annual average (1996 to 2000)

locations in Zones 1 and 2 where historically it was difficult to maintain a residual with chloramines. The number of "problem areas" was reduced by more than 75% during the trial. Booster chlorination facilities were added as part of the expansion of the Moore Reservoir in 2001, and it is expected that residual persistence in the Oakville system will improve further as a result.

CONCLUSIONS AND RECOMMENDATIONS

Water entering each system was considered to be of equal quality. A typical year of performance was available for the direct comparison of using free chlorine in the Oakville system with the Burlington system (year 2000 data).

Water quality was maintained or improved upon as a result of operating the Oakville system with free chlorine relative to both the historical use of chloramines in Oakville and the use of free chlorine in the adjacent Burlington system. The ability to maintain residuals throughout the distribution system was comparable for both systems in the year 2000, providing evidence of the successful use of free chlorine in the

Oakville system compared with the use of free chlorine in the Burlington system. In general, the bacteriological quality of the water in the Oakville system was maintained or enhanced when using free chlorine instead of chloramines. Further improvements are expected due to the startup of the new booster chlorination facilities at the Moore Reservoir.

More taste and odor complaints referencing chlorine were received in Oakville relative to Burlington as a result of the conversion from chloramines to free chlorine. An increase in complaints was observed during implementation, but complaints were received regularly throughout the trial period. A further increase in complaints of chlorinous taste and odor was observed during the fall of 2000 as modifications were made in how both systems were operated to meet the new requirements of the ODWS.

Operating the Oakville system with free chlorine resulted in a marginal increase in THMs (from 11 to 20 μg/L), but levels are still well below the ODWS of 100 μg/L.

It is recommended that the Region continue to operate the Oakville system with free chlorine for residual persistence.

Continued monitoring is recommended to allow for a comparison of performance over a longer time frame, as variations in performance were observed during the four-year period prior to the trial when using free chlorine.

In light of the performance of the Burlington system relative to the Oakville system, it is recommended that the Region review the operations of the Burlington system to allow the system to be operated to manage water quality.

The success of free chlorine for residual maintenance in the Oakville system suggests that the choice of secondary disinfectant needs to be made carefully: although there may be an increasing trend in the use of chloramines for residual maintenance, the use of free chlorine in lieu of chloramines should not be discarded as an option.

CHAPTER · 16

The Use of Decay Studies to Plan San Francisco's Conversion to Chloramines

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BACKGROUND

Chloramine decay studies were conducted to evaluate the impact of various water quality parameters on the decay characteristics of the San Francisco Public Utility Commission (SFPUC) water, to predict disinfectant levels within the distribution system, and to predict the potential for nitrification. The results of the studies show that overall, SFPUC water is very stable, meaning that decay rates are slow compared with other surface water supplies. The studies indicated that for SFPUC waters, pH had the most significant effect on chloramine decay; chloramine decay was slower at higher pH values (between pH 8.8 and 9.0), which are currently maintained for corrosion control.

The use of booster chlorination was also evaluated as a possible means of maintaining or increasing the total chlorine residual within the distribution system. Although chlorine added to chloraminated test water combined with the free ammonia-N that was liberated as a result of chloramine decay, the ability to properly feed sodium hypochlorite is a critical factor in determining whether or not to implement this practice. An evaluation of free ammonia-N data from a similar water system supported the conclusion that boosting the chloramine residual with sodium hypochlorite is not a practical alternative for the SFPUC system.

INTRODUCTION

Chloramine Decay and Nitrification

When chloramines decay, the total chlorine concentration decreases and free ammonia is released. Free ammonia can be converted to nitrite or nitrate through a biological process known as nitrification. Nitrification can cause several problems in a distribution system, the most significant of which is a rapid decrease in the total chlorine residual. For this reason, it is important to understand the rate of decay of chloramines and factors that influence the decay rate.

Disinfectant decay generally occurs in two phases. The first phase involves the oxidation of organic and inorganic matter and is often rapid, partly because the disinfectant concentration is at its highest level. A slower decay phase follows in which autodecomposition predominates.

San Francisco System

The SFPUC has three water sources. The primary source water is the Hetch Hetchy Reservoir in Yosemite National Park. Hetch Hetchy water is unfiltered, but it is treated with lime to adjust pH for corrosion control and is currently disinfected with free chlorine. The SFPUC's other two water sources are from local watersheds; these waters are filtered and treated at two water treatment plants, the Sunol Valley WTP (SVWTP) and the Harry Tracy WTP (HTWTP). The Hetch Hetchy water is typically blended with water from the Sunol Valley WTP. The SFPUC source waters (the blend and HTWTP effluent) are typically low in TOC (1.5 mg/L on average) and have high pH (between 8.5 and 9.0). The SFPUC proceeded to construct the required chemical treatment facilities to chloraminate its water supplies (Chloramine Conversion Project). Chloramines would be formed in SFPUC waters after achievement of *CT* with free chlorine.

San Francisco's distribution system includes 12 major storage reservoirs, which range in size from 1 MG to 89 MG in capacity. The reservoirs were originally designed to maximize storage and guarantee adequate fire flow. Accordingly, excessive reservoir detention time and poor mixing are of particular concern for San Francisco, especially in its larger reservoirs. San Francisco's projected conversion to chloramines as the residual disinfectant heightened concerns regarding water age and the potential for nitrification.

OBJECTIVES

An understanding of chloramine decay is important to effectively manage chloramine residuals in the distribution system. This understanding is useful to interpret distribution system total chlorine residual data and to make decisions regarding treatment targets and distribution system operations, such as flushing, reservoir operation, and booster chlorination.

Several studies were conducted from 1998 to 2001 to quantify the chloramine decay rates for the various SFPUC source waters, to evaluate the impacts of water quality parameters on chloramine decay, and to evaluate the potential use of chlorine boosting as a means of preventing nitrification.

IMPACT OF WATER QUALITY PARAMETERS ON CHLORAMINE DECAY

Studies were conducted to evaluate the impact of source water quality and other parameters on chloramine decay. The study evaluated four possible source water combinations: SVWTP, Hetch Hetchy, Hetch Hetchy and SVWTP blend, and HTWTP. The study compared the effects of pH, temperature, total chlorine dose, and chlorine–to–ammonia-N ratio on the decay rate and other chloramine parameters.

Results

The results of the studies were consistent with the literature. According to the literature, higher TOC, higher temperature, higher alkalinity, and lower pH yield faster decay rates. SVWTP water with its higher TOC was shown to decay faster than the blend with Hetchy or HTWTP water. However, the studies concluded that because TOC is typically low in SFPUC's waters, pH is the most significant factor influencing chloramine decay in both the Hetch Hetchy/SVWTP blend and the HTWTP waters.

The studies confirmed that a higher initial total chloramine dose (3 mg/L) resulted in a faster decay rate compared with a lower chloramine dose (1 mg/L). The studies also demonstrated that a lower chlorine:ammonia-N ratio (3:1) resulted in excessive amounts of free ammonia and should be avoided to prevent nitrification.

The SFPUC currently has a system target of pH 9.0 for corrosion control; these studies support maintaining this pH to optimize chloramine treatment and minimize decay in the distribution system. The studies also support the selection of an initial total chlorine dose of 2.0 mg/L.

This dose should minimize taste and odor complaints, minimize chloramine decay, and provide an adequate residual in the distribution system. Based on the studies, a chlorine:ammonia-N ratio of 4.7:1 was also recommended. This ratio will result in monochloramine as the predominant species and will minimize the initial amount of free ammonia-N in the distribution system; at this ratio the free ammonia-N will be 0.05 mg/L or less.

EVALUATION OF CHLORINE BOOSTING

During the conceptual engineering phase of the Chloramine Conversion Project, the concept of implementing booster chlorination in the chloraminated distribution system was highlighted. The purpose would be to add free chlorine to combine with the free ammonia (liberated due to chloramine decay), forming chloramines, thereby increasing or "reestablishing" the total chlorine residual. Since ammonia-N is the primary nutrient responsible for nitrification, this operational practice was viewed as a potential action for the prevention of nitrification.

The goals of this evaluation (conducted during summer 2001) were to determine the decay rate for water in which the total chlorine residual has been boosted and compare that decay rate with the decay rate of a split sample of the same water without a boosted residual.

Two tests were conducted; in both tests the target total chlorine residual was 2.5 mg/L. The first test had an initial target chlorine–to–ammonia-N ratio of 3:1, representing a possible chemical misfeed at the treatment plant and introduction of excess ammonia-N into the system. The second test had a target ratio of 4.5:1, which is near the theoretical optimum ratio for the formation of monochloramine and would ensure the presence of free ammonia-N. Both tests were conducted on the blend of Hetch Hetchy and SVWTP water. For both tests, the chloramine residual was allowed to decay for 38 days before being boosted by the addition of sodium hypochlorite. The booster chlorine doses were calculated based on the measured free ammonia-N. For both tests, chlorine was added at the target ratio of 4.5:1.

Results

Table 1 shows the total chlorine, free ammonia, and pH for the boosted and nonboosted waters for both tests at day 66 of the experiment.

This study shows that boosting the total chlorine residual with sodium hypochlorite reduces the free ammonia-N concentration. However, it is important to note that even in the nonboosted water, the total chlorine residual remained above 1.5 mg/L after 66 days. These data highlight the stability of SFPUC water. Figure 1 shows the decay curves and production of free ammonia-N during the study.

All of the results support the general conclusion that chloramines are very stable in SFPUC source waters. The tests suggest that with an

Table 1 Total chlorine residual, free ammonia-N, and pH after approximately two months

Test Condition	Treatment	Total Chlorine	Free Ammonia-N	pH
Test 1: Initial ratio 3:1	Nonboosted Boosted	1.8 mg/L 2.3 mg/L	0.20 mg/L 0.08 mg/L	9.2 9.1
Test 2: Initial ratio 4.5:1	Nonboosted Boosted	1.55 1.61	0.15 0.07	8.83 8.56

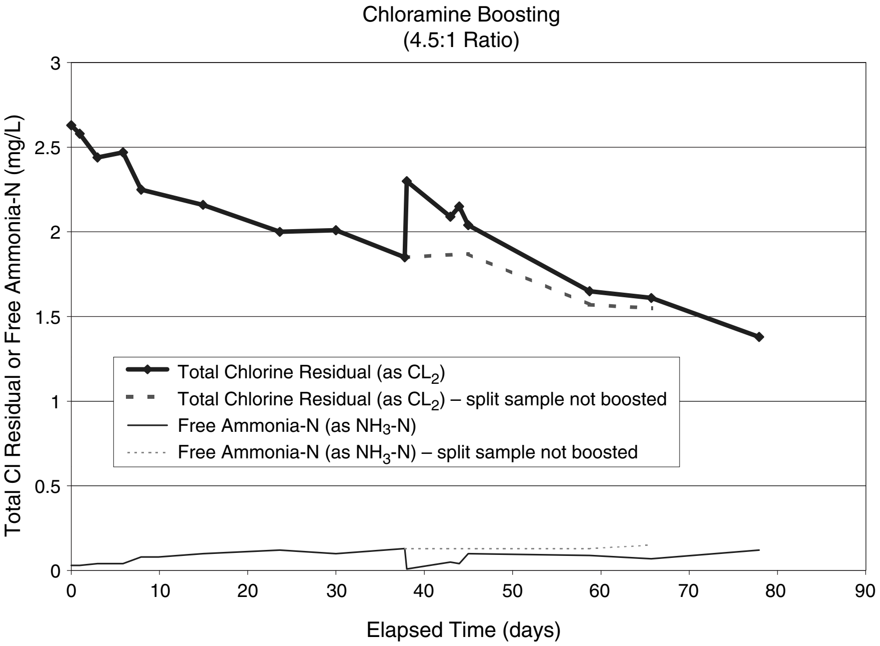

Figure 1 Total chlorine decay and production of free ammonia-N

initial dose of 2.0 mg/L, the expected total chlorine concentration in the distribution system will be above 1.5 mg/L after 30 days and the total ammonia-N concentration is expected to be below 0.2 mg/L. The experience of another utility with similar source water indicates that maintaining a minimum residual of 1.5 mg/L in the distribution system can minimize the occurrence of nitrification (Figure 2). The SFPUC is using 30 days as an upper limit for maximum detention times in its largest reservoirs and is evaluating alternatives to promote turnover and mixing in those reservoirs.

FULL-SCALE EVALUATION OF SIMILAR WATER

Since the chloramine decay studies of SFPUC water could only be performed at bench scale, data from a similar water utility's chloraminated distribution system were evaluated. The evaluation examined approximately 125 samples for which total chlorine residual, free ammonia-N, and nitrite-N data were available. Figure 3 shows a scenario in which a free ammonia-N concentration is calculated based on the initial total chlorine residual, the measured total chlorine residual in the distribution system, and the initial ratio used to generate chloramines (to determine the initial free ammonia-N). This calculated free ammonia-N is multiplied by 4.5 to determine the amount of sodium hypochlorite to add to the water to boost the residual. Next, a ratio is determined using the calculated concentration of chlorine and the actual or measured free ammonia-N concentration. For all of the samples, the resulting chlorine–to–ammonia-N ratio would be greater than 4.5:1 because the actual free ammonia-N concentration is much lower than the expected (or calculated) free ammonia-N concentration. This scenario shows that it is inappropriate to boost chloraminated water based on the total chlorine residual values measured in the distribution system alone.

Figure 4 compares the observed total chlorine residual with the expected total chlorine residual if booster chlorine is added based on the observed free ammonia-N concentration. For example, if a sample has 0.1 mg/L free ammonia-N and 4.5 times as much, or 0.45 mg/L, of chlorine is added, the total chlorine residual will be increased or boosted by 0.45 mg/L. If 0.05 mg/L of free ammonia-N is present and 4.5 times as much, or 0.225 mg/L, of chlorine is added, the total chlorine residual will be increased by 0.225 mg/L. Figure 4 implies that where the measured total chlorine residual is low in the distribution system, free ammonia-N will be unavailable for combining with chlorine to boost the residual.

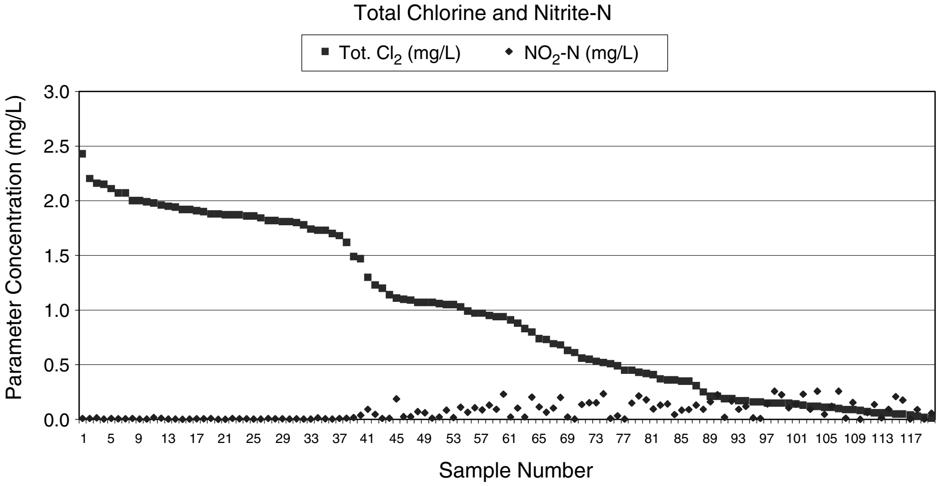

Figure 2 Similar full-scale distribution system total chlorine and nitrite-N data

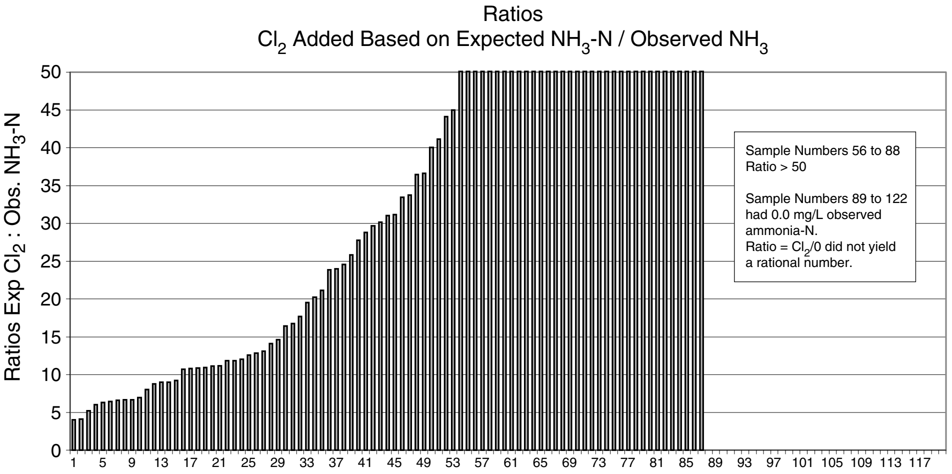

Figure 3 Ratio of chlorine (calculated) to ammonia-N (actual)

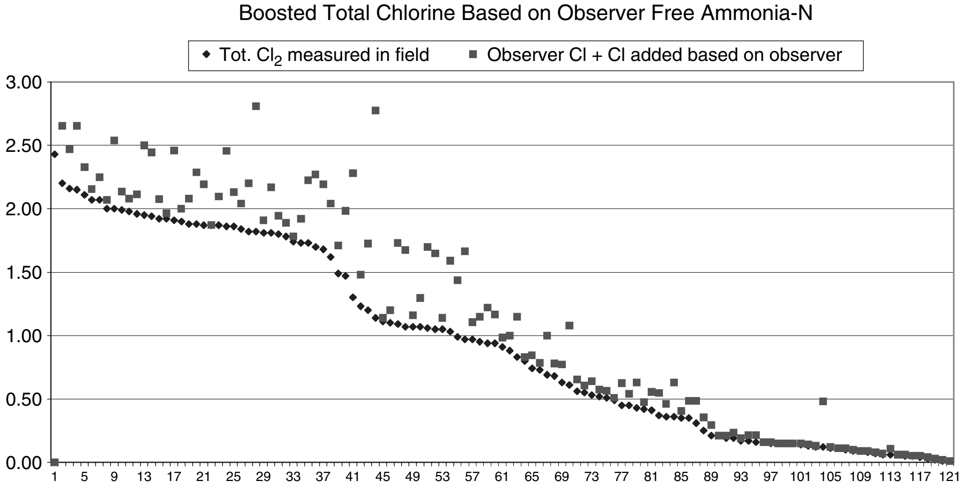

Figure 4 Expected final total chlorine residual when chlorine added is based on ammonia-N measured

IMPLICATIONS FOR FULL-SCALE IMPLEMENTATION IN SAN FRANCISCO

The bench-scale studies on booster chlorination demonstrated that it is possible to recombine free ammonia-N that is liberated through the chloramine decay process. Since ammonia-N is the nutrient responsible for nitrification, the practice of chlorine boosting may be viewed as an action for the prevention of nitrification. However, data from a utility with similar water quality indicate that where the total chlorine residual is low and boosting may be of benefit, sufficient free ammonia-N may not be available to combine with the chlorine added to boost the residual. Furthermore, full-scale experience from other utilities indicates that once nitrification is under way, boosting the chloramine level is not an effective control strategy.

Precise monitoring and operational controls must be available to implement booster disinfection. Five elements are required for successfully boosting the total chlorine residual:

- Accurate measurement of free ammonia-N (mg/L)
- Accurate measurement of water flow (gallons per minute)

- Ability to feed sodium hypochlorite at the correct dosage (chlorine–to–ammonia-N ratio) and proportional to the measured flow
- Proper mixing of sodium hypochlorite solution after injection
- Downstream total chlorine monitoring capabilities

The equipment to accomplish these requirements is expensive and requires significant operator commitment for proper maintenance and calibration. If these features are not in place, there is a high probability of misfeeding chlorine, which could result in the formation of dichloramine or trichloramine (according to the breakpoint chlorination curve), resulting in low total chlorine residuals and possibly taste and odor problems. Before making the decision to implement booster disinfection, the SPFUC must assess the risk of applying an improper chlorine dose and the possible benefit of reducing free ammonia-N concentration. Some utilities practice the addition of chlorine and ammonia to boost a total chlorine residual. This option is currently under consideration by the SFPUC for certain areas of the distribution system.

ACKNOWLEDGMENTS

The authors would like to acknowledge Bianca Joy Ferreros, Hsiao-Lung Chang, and Patrick Lo for their technical and analytical contributions to the various studies conducted over the years, and for their overall laboratory support. In addition, Bianca Joy Ferreros, Katie Miller, Jackie Cho, and Alan Wong contributed to the management and review of the more recent project studies.

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CHAPTER · 17

Distribution System Water Quality Following Blending Surface, Ground, and Saline Sources

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BACKGROUND

The University of Central Florida is conducting a \$3 million Tailored Collaboration Project to determine the effects of blended water quality on distribution system water quality. Finished waters from seven different processes are blended in varying ratios before distribution to 18 pilot distribution systems (PDSs) in a facility that includes 8,500 ft^2 of roofing and 4,500 ft^2 of cement pad. The facility is operated continuously. Processes include conventional groundwater, softening, enhanced surface water, integrated membrane system, and reverse osmosis treatment. The PDSs were made from galvanized, cast iron, lined cement, and PVC pipe taken from actual distribution systems. Findings are:

1. Alkalinities lower than background (200 mg/L $CaCO_3$) have resulted in release of apparent color to 50 cpu. Apparent color release is due almost totally to particulate iron and appears independent of pH, LSI, calcium, and all other parameters associated with $CaCO_3$ indexes except for alkalinity. Color release is also positively correlated with chloride and sulfate concentrations.
2. Color release has continued through 6 months of equilibration and 15 months of field testing from CI pipes receiving less than background alkalinity.
3. Total and free chlorine dissipation was accurately predicted by modeling residual as a function of pipe material, UV-254,

temperature, and time. Field data and model projections have shown:

   a. The dissipation rate of total or free chlorine is approximately an order of magnitude greater in galvanized and CI pipe than in lined cement and PVC pipe.
   b. Maintenance of required residuals in galvanized and CI pipe is practically impossible in summertime conditions.
   c. pipe material is the most significant factor affecting residual maintenance for expected water quality conditions in distribution systems.

4. Ferric hydroxide and cupric hydroxide appear to be the controlling solid films based on surface and equilibrium analyses.
5. Biological stability did not change as a result of film disruption as indicated by apparent color release. Field investigations will continue until June 2003.

INTRODUCTION

The University of Central Florida (UCF) is conducting a $3 million Tailored Collaboration Project (TCP) for Tampa Bay Water (TBW) to determine the effects of changing finished water quality on distribution system water quality. TBW manages drinking water resources for six member governments (MG) on the central west coast of Florida. These MGs are the cities of Port St. Richey, St. Petersburg, and Tampa, and Hillsborough, Pasco, and Pinellas Counties. Groundwater has been historically used for MG drinking water supply; however, desalination of seawater and enhanced treatment of surface water have to be utilized for future supply. Currently TBW has a 60-mgd enhanced surface water treatment plant (CSF-O_3-GAC) and a 25-mgd desalination plant in operation. A second 25-mgd desalination plant is scheduled for design and construction. These waters with groundwater are distributed to the MG in a large pipeline that runs through a three-county area, and vary in water quality. TBW and the MGs have defined the acceptable water quality in a control document known as Appendix D. The average water quality from the three primary water supply sources is shown in Table 1. Although water quality by source has been defined, there is significant water quality variation among sources. Consequently, TBW and AwwaRF

Table 1 Average water quality for three primary TBW MG finished waters

Parameter	Units	Groundwater	RO Water	Surface Water
pH		7.6	8.3	8.3
TDS	mg/L	400	150	300
HCO_3	mg/L as $CaCO_3$	225	50	50
Ca	mg/L as $CaCO_4$	250	30	50
Mg	mg/L as $CaCO_5$	25	20	20
Na	mg/L	10	30	15
Cl	mg/L	15	50	10
SO_4	mg/L	10	30	180
Si	mg/L as SiO_2	20	20	10
TOC	mg/L C	3	0.3	3
NH_2Cl	mg/L Cl_2	4	4	4
HPCs	cfu/mL	50	50	100
AOC	µg/L	100	50	150
PEPA	#/cm^2			

have sponsored UCF in a TCP to determine the effects of blended water quality on distribution system water quality.

Pilot plants that are similar to the existing and future TBW and MG water treatment processes simulate the water quality. All of the processes have disinfection and pH stabilization for posttreatment. Finished waters from seven different processes are blended in varying ratios before distribution to 18 pilot distribution systems (PDSs) in a facility that includes 8,500 ft^2 of roofing and 4,500 ft^2 of cement pad. The pilot plant facility consists of a surface water stainless-steel trailer, raw water storage tanks, process tanks, a PDS, cradles, and a corrosion shed. The water is distributed to 18 PDSs through a manifold, which prevents water loss from the PDS. The facility is operated continuously. Cradles containing coupons used for biofilm and surface analyses, and a corrosion pilot containing copper pipe with lead coupons follow each PDS. The PDSs were made from galvanized, cast iron (CI), lined cement, and PVC pipe taken from actual distribution systems.

APPARENT COLOR RELEASE

The 18 PDSs exhibited a significant release of apparent color, which was directly correlated to total iron, as shown in Figure 1. There was excellent correlation among apparent color, turbidity, and total iron; however, the apparent color was due to release of particulate iron matter from unlined iron-containing pipes, as shown in Figure 2. There was no release of apparent color from the lined cast iron (LCI) and PVC pipe.

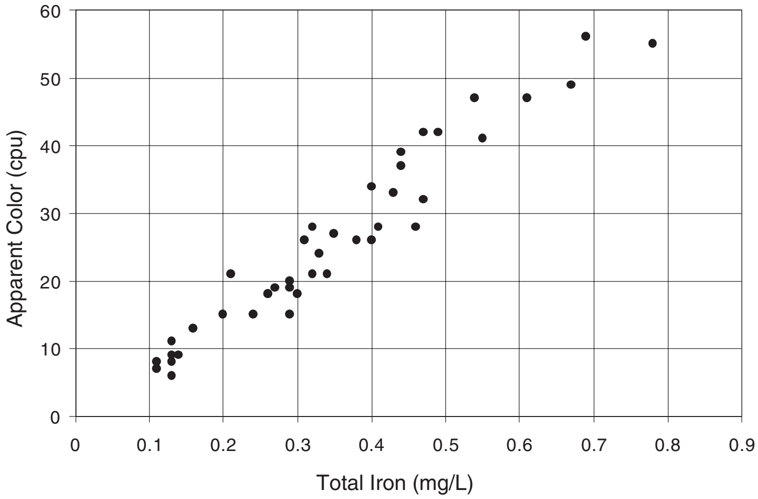


Figure 1 Apparent color versus total iron

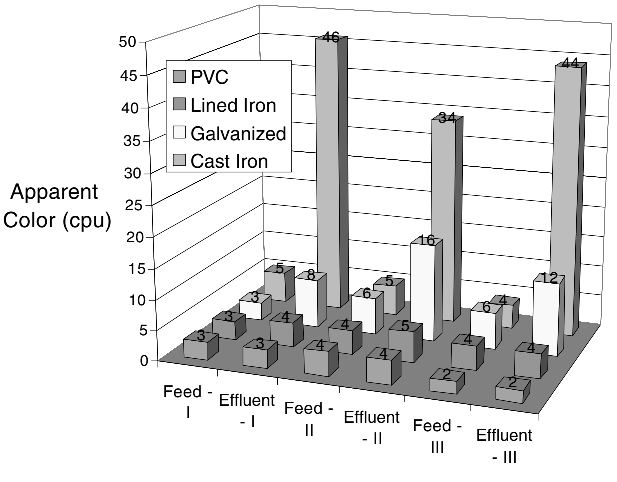


Figure 2 Apparent color of feed and effluent PDS streams for varying pipe material

A zero-order model was developed to describe the release of apparent color, as shown in Equation 1. Apparent color, total iron, or turbidity can be interchanged in the model because of the correlation among them. K_m is the flux term (mass/area-time) and is a function of Reynolds number and water quality. The model for iron release was used to develop the projected iron release as a function of Reynolds number for varying process and blended water quality, as shown in Figure 3. The most significant water quality variable affecting the release of total iron, apparent color, or turbidity is alkalinity; however, Na, Cl, SO_4, DO, and temperature can impact release. The model predicts that any alkalinity lower than background groundwater alkalinity will cause some release of turbidity, apparent color, and total iron.

$$C_t = C_0 + \frac{4}{D}K_m\Delta t \qquad (1)$$

where:

$$K_m = b_1(\text{Re}) + b_2(\text{WQ})$$

These data are limited to large HRT (two or five days). Found pipe material was significant; cast iron and galvanized experienced greater release than lined iron or PVC. Source water characteristics were also important, as historical sources (G1 and G2) did not exhibit film release. Source waters with lower alkalinity than historical groundwater source exhibited release, which included S1, RO, and blends of S1 and RO. The response to blend change was rapid, persistent, and detected in a matter

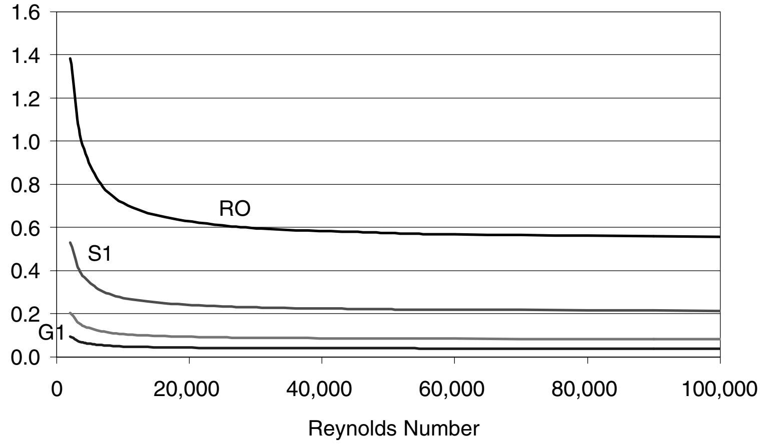


Figure 3 Iron release for varying water sources versus Reynolds number

of days. Elevated values remain 14 months after introduction of the new blend. The principal form of iron and apparent color is particulate, as correlation among apparent color, total iron, and turbidity was excellent. Color release was impacted by water quality associated with treatment in the following order: RO > S1 > G1. An empirical water quality model was developed for use with any source blend, which models iron flux as a function of water quality and Reynolds number.

IRON, COPPER, AND LEAD RELEASE

Iron

Theoretical dissolution/precipitation models were developed according to the solubility of controlling solid phase by which the iron ion concentration in water is determined. Specifically separate models are developed for various solid phases based on thermodynamic equilibrium, which include $FeCO_3$, $Fe(OH)_2$, and $Fe(OH)_3$. These models are developed based on the assumption that ORP = 0.38V (average of TBW data) and temperature = 25°C, and are shown in Equations 2, 3, and 4.

1. $FeCO_3$ Model

$$
\begin{aligned}
Fe_T = \frac{Kso \times [H^+]}{Alka \times Ka_2}\Big(1 + \frac{10^{-9.5}}{H^+} + \frac{10^{-20.6}}{H^{+2}} + \frac{10^{-32}}{H^{+3}} + 100Alka \\
+ 10^{4.38}Alka \times \frac{Ka_2}{[H^+]} + 10^{2.25}[SO_4^{-2}] + 10^{0.9}[Cl^-]\Big)
\end{aligned}
\tag{2}
$$

2. $Fe(OH)_2$ Model

$$
\begin{aligned}
Fe_T = \frac{Kso}{[(OH)^-]^2}\Big(1 + \frac{10^{-9.5}}{H^+} + \frac{10^{-20.6}}{H^{+2}} + \frac{10^{-32}}{H^{+3}} + 100Alka \\
+ 10^{4.38}Alka \times \frac{Ka_2}{[H^+]} + 10^{2.25}[SO_4^{-2}] + 10^{0.9}[Cl^-]\Big)
\end{aligned}
\tag{3}
$$

In these models, the free iron ion concentration was first calculated assuming there was equilibrium between iron ion and controlling solid phase. Then the concentrations of other dissolved complexes were estimated according to the related reactions. The sum of dissolved iron ion and dissolved complexes is the total iron concentration in bulk water, as shown in the previous equations. According to the models, it is evident

3. $Fe(OH)_3$ Model

$$\begin{aligned} Fe_T = {} & 3\times 10^{3.83}[H^+]^5 + 2\times 10^{2.39}[H^+]^4 + 10^{3.2}[H^+]^3 \\ & + 10^{0.15}[H^+]^2 + 10^{-3.11}[H^+] + 10^{-10.6} + \frac{10^{-19.5}}{[H^+]} \\ & + 10^{7.24}[H^+]^3[SO_4^{-2}] + 10^{8.58}[H^+]^3[SO_4^{-2}]^2 \\ & + 10^{4.68}[H^+]^3[Cl^-] + 10^{5.33}[H^+]^3[Cl^-]^2 \\ & + 10^{4.33}[H^+]^3[Cl^-]^3 + 10^{9.8}[H^+]^3(1 + \frac{10^{-9.5}}{[H^+]} \\ & + \frac{10^{-20.6}}{[H^+]^2} + \frac{10^{-32}}{[H^+]^3} + 100Alka + 10^{4.38}Alka + \frac{Ka_2}{[H^+]} \\ & + 10^{2.25}[SO_4^{-2}] + 10^{0.9}[Cl^-]) \end{aligned} \tag{4}$$

that both pH and alkalinity have significant effects on iron release. Iron release will decrease with increasing pH and/or alkalinity. Based on the $Fe(OH)_2$ and $Fe(OH)_3$ models, pH significantly affects iron release while alkalinity does not. The $Fe(OH)_2$ model overpredicts iron release. The predicted values are 1,000 times higher than actual TBW pilot total iron data. The $Fe(OH)_3$ model underpredicts iron release. The predicted values are 200,000 times lower than actual TBW pilot dissolved iron data. The $FeCO_3$ model more accurately predicts iron concentration than the $Fe(OH)_2$ and $Fe(OH)_3$ models and is shown in Figure 4. The predicted values are a little higher than actual dissolved iron data and a little lower than actual total iron data. Predicted soluble iron varies with temperature. If temperature increases, solubility of $Fe(OH)_2$ and $Fe(OH)_3$ will increase, so predicted values will increase. Solubility of $FeCO_3$ will decrease when temperature increases.

The pH, alkalinity, chloride, and sulfate can affect iron release according to the model. The sensitivity of iron was assessed using the average water quality of the parameters from the pilot study (pH 8.0, alkalinity 110 mg/L, Cl^- 48.45 mg/L, SO_4^{-2} 68.5 mg/L).

1. The sensitivity of iron to pH is (–250 μg/L iron)/pH from pH 7.5 to 8.5, and to alkalinity is (–70 μg/L iron)/(100 mg/L alk.) from 50 to 200 mg/L $CaCO_3$. These sensitivities increase at lower pH or alkalinity.

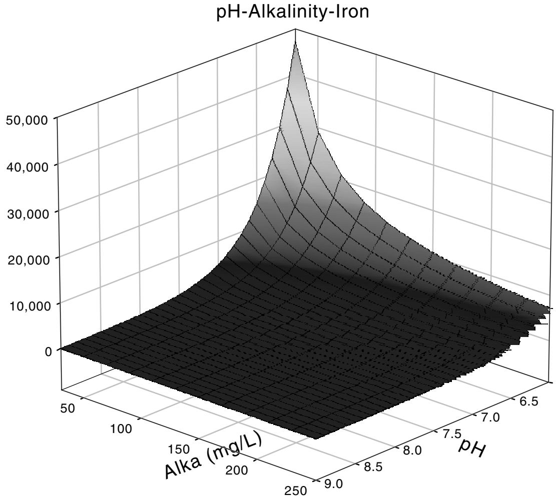


Figure 4 Iron concentration as a function of alkalinity and pH based on siderite as controlling solid phase

2. The sensitivity of iron to chloride is (–1.35 μg/L iron)/(100 mg/L chloride) and to sulfate is (–11.2 μg/L iron)/(100 mg/L sulfate). These two sensitivities are constant.
3. pH and alkalinity significantly affect iron release. Sulfate and chloride affect iron release but respectively less than pH or alkalinity.

Surface characterization was conducted on cast iron, galvanized steel (G), LCI, and PVC coupons. These coupons were placed in and taken out of the cradles every three months and were made from the pipes taken from the MG distribution systems. Iron deposits were the most significant deposits found. The primary corrosion products of the initial coupons from the CI pipe were α and β-FeOOH, which was converted to γ-Fe_2O_3 over the three-month period. $CaCO_3$ was also found in the final corrosion product. The major corrosion products of the initial G coupons were β-FeOOH and ZnO, while Fe_2O_3 was the major product for the final coupons. No obvious crystal phases were found on initial LCI coupons, but $CaCO_3$ and γ-Fe_2O_3 were found on the final LCI coupons. Hence the surface characterization showed γ-Fe_2O_3 was the major corrosion product for iron metal-based pipes.

Copper

Copper levels in drinking water can be predicted based on dissolution/precipitation of various solid phases resulting from copper corrosion. For cupric species, solid phases are primarily associated with carbonate and hydroxide. Sulfate and chloride ions also affect solid phases, although their complexes may not be significant compared with those mentioned above. In this study, theoretical models are developed separately based on the solid phase controlling the minimum cupric concentration at which the corresponding precipitation happens. Specifically, separate models are developed for various solid phases based on thermodynamic equilibrium; they included tenorite and cupric hydroxide. Malachite could become a controlling phase under extremely high alkalinity, which is not in the range of typical drinking water, so no modeling efforts were made for malachite.

In these models, the cupric ion concentration is first calculated assuming it is at equilibrium with controlling solid phase. Then the concentrations of other dissolved complexes are estimated and the sum of dissolved cupric ion and dissolved complexes provides total copper level in bulk water, as shown in Equations 5 and 6.

1. Tenorite (CuO) Control

$$\begin{aligned}\text{DICu} = {} & 10^{-8.26} + 10^{0.02}[H^+] + 10^{-18.92}[H^+]^{-1} \\ & + 10^{20.11}[H^+]^3[CO_3^{2-}] + 10^{14.8}[H^+]^2[CO_3^{2-}] \\ & + 10^{18.58}[H^+]^2[CO_3^{2-}]^2 + 10^{-5.16}[CO_3^{2-}] \\ & + 10^{7.98}[H^+]^2 + 10^{8.38}[H^+]^2[SO_4^{2-}]\end{aligned} \tag{5}$$

2. Cupric Hydroxide ($Cu(OH)_2$) control

$$\begin{aligned}\text{DICu} = {} & 10^{-7.35} + 10^{0.93}[H^+] + 10^{-18.01}[H^+]^{-1} \\ & + 10^{20.01}[H^+]^3[CO_3^{2-}] + 10^{15.71}[H^+]^2[CO_3^{2-}] \\ & + 10^{19.49}[H^+]^2[CO_3^{2-}]^2 + 10^{-4.25}[CO_3^{2-}] \\ & + K_{13}[H^+]^2 + 10^{9.29}[H^+]^2[SO_4^{2-}]\end{aligned} \tag{6}$$

Experimental data obtained from the pilot study were compared with theoretical values predicted by two different models. Both the tenorite and cupric hydroxide models underpredict actual copper concentration. This could be explained by particulate copper release, which is not considered in the equilibrium models. The impact of sulfate and chloride on iron solubility is significantly lower than the effect of pH and alkalinity. Sulfate and chloride have negligible effects on copper release compared with pH and alkalinity.

The data from the PDS were correlated to Cu concentrations from the copper lines in the corrosion shed. Both nonlinear and linear regressions of more than 28 water quality variables were done. The following model (Equation 7) was the best for predicting actual copper concentration and shows only alkalinity and pH as significant water quality variables affecting copper concentration.

$$\text{Cu (mg/L)} = 7.18 + 0.0034 \times \text{Alk} - 0.85 \times \text{pH} \qquad (7)$$

The effects of blends of the three primary sources of TBW finished water are shown in Figure 5 using the copper model and typical water quality shown in Table 1. The copper model predicted violation 1.3 mg/L 90th-percentile action level for blends of finished water that contained more than 85% conventional groundwater (G1, HCO_3 225 mg/L as $CaCO_3$). Hence, member governments (MG) that utilize high alkalinity

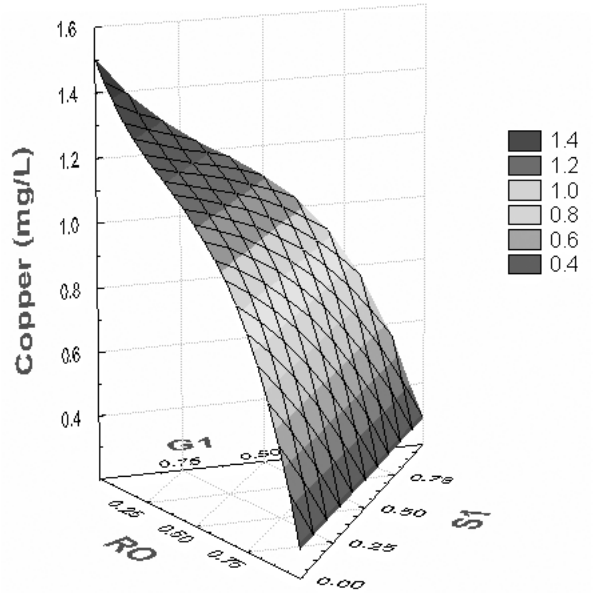


Figure 5 Predicted copper concentration for varying blends of reverse osmosis, surface water, and groundwater

sources will need supplemental corrosion control (antiscalents) to meet the copper and lead rule action levels.

Lead

A thermodynamic model for dissolved inorganic lead (DIPb) was developed based on hydrocerussite as the controlling solid phase, and is shown in Equation 8 and in Figure 6 as a function of pH and alkalinity.

$$\begin{aligned}\text{DIPb} = {} & [Pb^{2+}] + 4[Pb_4(OH)_4^{4+}] + 2[Pb_2OH^{3+}] + [PbOH^+] \\ & + 3[Pb_3(OH)_4^{2+}] + 6[Pb_6(OH)_8^{4+}] + [Pb(OH)_2^0] \\ & + [Pb(OH)_3^-] + [Pb(OH)_4^{2-}] + 3[Pb_3CO_3^{4+}] \\ & + 2[Pb_2CO_3^{2+}] + [Pb(CO_3)_2^{2-}] + [PbHCO_3^+] \\ & + [PbSO_4^0] + [Pb(SO_4)_2^{2-}] + [PbCl^+] + [Pb(Cl)_2^0] \\ & + [Pb(Cl)_3^-] + [Pb(Cl)_4^{2-}]\end{aligned} \tag{8}$$

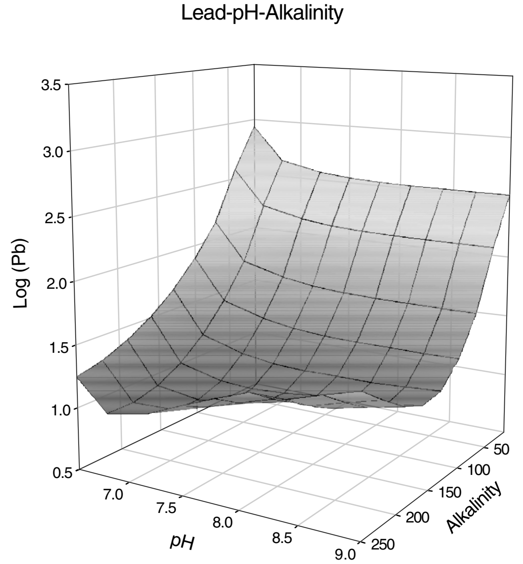


Figure 6 Thermodynamic model for lead based on hydrocerussite as controlling solid phase

The model predicts that lead solubility will increase with decreasing alkalinities at the pH and alkalinities that will be common to the MG distribution systems. However, this model did not predict the observed lead concentrations from the corrosion shed primarily because of the large contribution of particulate lead.

Weekly, biweekly, and monthly water quality monitoring of 27 different parameters in the 18 PDS corrosion lines found: (1) higher sulfate, SiO_2, and calcium decreased lead levels; and (2) lead release increased with increasing chloride and there was no obvious effect of alkalinity, pH, DO, conductivity, and temperature on lead release. A nonlinear lead model was developed using nonlinear regressions and is shown in Equation 9. The data collected from the 18 lines in the copper and lead corrosion shed were used to develop the empirical model.

$$\begin{aligned}\text{Lead} = {} & 1.045^{T-25}(-0.102 \times \text{Alkalinity} - 0.74 \times \text{pH} \\ & + 0.13 \times \text{Chloride} - 0.17 \times \text{Sulfate} \\ & + 0.07 \times \text{Conductivity} + 1.69 \times \text{DO})\end{aligned} \tag{9}$$

The empirical total lead model as shown in Equation 9 was used to develop a relationship of total lead to varying blends of desalination, surface water and groundwater and the information shown in Table 1. The graphical model shown in Figure 7 indicates that 12 μg/L will be the highest observed lead concentration for any blend of desalination, surface water and groundwater, and no violation of the lead action level.

The thermodynamic equilibrium model (hydrocerussite) did not predict lead release observed in the pilot study. The regression model showed that increasing pH, alkalinity, sulfate, and SiO_2 decreased lead release. The model also projected that higher chloride, temperature, and conductivity increased lead release. The lead model predicted no violation of the 15 μg/L 90th-percentile action level at a temperature of 25°C. However, the lead violation increased with increasing temperature.

RESIDUALS

Models for free chlorine and chloramine dissipation were developed using the data obtained for the different waters and pipe materials. These models are developed for the PDS used for the pilot study and should not be used to describe residual for pipe diameters larger than 6 inches. The parameters evaluated were pipe material, chlorine dose (mg/L), temperature (°C), time (hr), and UV254 (cm^{-1}, a TOC surrogate). The models

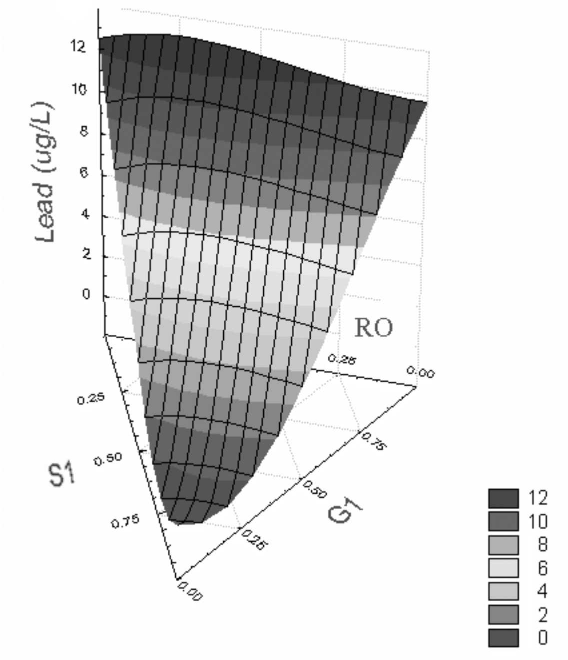


Figure 7 Predicted lead concentration for varying blends of reverse osmosis, surface water, and groundwater

were developed using nonlinear regression and verified using independent data. The free chlorine and chloramine models were verified using independent data. The models and the coefficients for the models are shown in Equations 10, 11, and 12 and Tables 2 and 3.

$$Cl_{2-t} = Cl_{2-0} \times \exp(-K \times UV254 \times A^{T-20} \times t) \qquad (10)$$

Free chlorine model

where:

Cl_{2-t} = chlorine concentration at time t (mg/L as Cl_2)
Cl_{2-0} = initial chlorine concentration (mg/L as Cl_2)
K = kinetic constant, cm/hr
A = temperature correction coefficient
T = temperature in degrees Celsius
t = time expressed in hours

Table 2 Parameters for free chlorine model

Material	Parameter		R2
	K	0.0723	
PVC	A	1.2343	0.98
	K	0.1483	
Lined iron	A	1.0917	0.93
	K	0.4442	
Cast iron	A	1.085	0.93
	K	1.517	
Galvanized	A	1.1535	0.93

$$Cl_{2-t} = Cl_{2-0} \times \exp(-(K_8 \times UV254 + K_w) \times A^{T-20} \times t) \qquad (11)$$

Chloramine residual model for PVC, lined cast iron, and cast-iron pipe

Table 3 Parameters for combined chlorine model

Material	Parameter		R2
PVC	K_B	0.0331	
	K_W	0.0012	
	A	1.1098	0.93
Lined iron	K_B	0.0263	
	K_W	0.0012	
	A	1.1231	0.92
Cast iron	K_B	0.1796	
	K_W	0.0121	
	A	1.1111	0.87

$$Cl_{2-t} = Cl_{2-0} \times \exp(-K \times A^{(T-20)} \times t) \qquad (12)$$

Chloramine residual for galvanized steel pipe

where:

K = overall decay constant = 0.0772 hr^{-1}
A = temperature correction coefficient = 1.0286
Cl_{2-t} = chlorine concentration at time t (mg/L as Cl_2)
Cl_{2-0} = initial chlorine concentration (mg/L as Cl_2)
T = temperature in degrees Celsius
t = time expressed in hours

The model for free chlorine is first order and shows that chlorine decay is highly affected by the pipe material. The rates of free chlorine dissipation for PVC and line iron are 0.07 cm/hr and 0.15 cm/hr, respectively, and 3–6 and 10–20 times greater for cast iron and galvanized steel pipes, respectively. The models show that the rate of chlorine dissipation also increases with an increase of either temperature or organic content. Doubling the organic concentration will double the rate of chlorine dissipation. The values of the coefficient for temperature indicate that the rate of free chlorine decay will double for an increase of 4°C for PVC and galvanized steel and every 8°C for lined and unlined cast iron.

Processes that produce water with a low organic content, such as RO and NF, provide an advantage because a reduction of 50% in TOC increases the time that the chlorine residual can be maintained in a 100% cast-iron or galvanized steel pipeline. Free chlorine residual for all of the pipe material is shown in Figure 8 for varying time and temperature.

Chloramine decay follows first-order kinetics and is modeled using two constants: one for the bulk liquid reactions, K_B, and another for the pipe wall reactions, K_W. K_B for PVC and lined iron is 0.0331 and 0.0263 cm/hr, respectively. For cast iron it is 6–8 times greater with values of 0.1796 cm/hr. K_W is also 1 order of magnitude greater for cast iron than for PVC and lined iron. A simpler first-order model was used for galvanized steel as shown in Equation 12, due to a reduced database. The rate of chloramine decay doubles for every 6°–7°C increase in temperature. Figure 9 shows total chlorine residual for different pipe materials. In general, PVC and LCI pipes have the capacity to keep chlorine residual longer, under the same conditions, than the other materials. The iron-based pipes show a rapid decline in the chlorine concentration due to the reaction of chlorine with the corrosion products of iron.

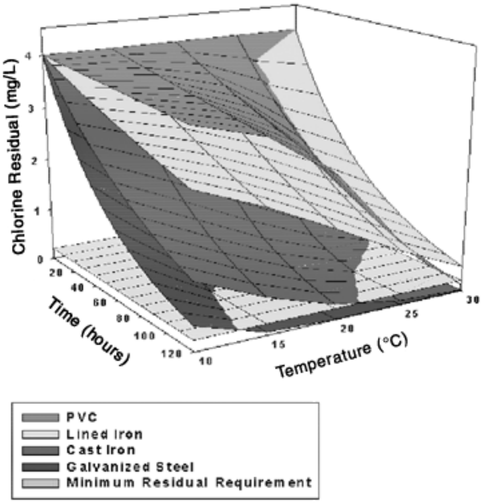


Figure 8 Effect of time and temperature on free chlorine residual for varying pipe materials

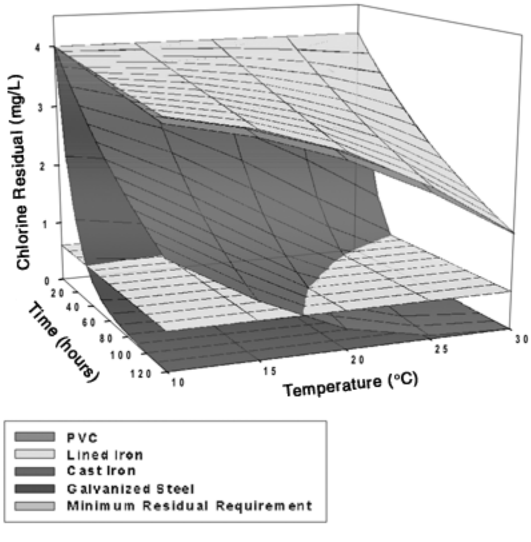


Figure 9 Effect of time and temperature on combined chlorine residual for varying pipe materials

Chloramine residuals are difficult to maintain in summertime conditions regardless of the disinfectant utilized. Assuming summertime conditions as 3 mg/L TOC, pH 8, 28°C, and a 4 mg/L NH_2Cl residual, the time to reach 0.6 mg/L NH_2Cl for CI, LCI, PVC, and G pipes would be 1.2, 8.9, 8.4, and 0.8 days, respectively. In the case of free chlorine, the respective times would be 1.7, 4.7, 3.6, and 0.3 days. These models indicate that MG will have difficulty maintaining residuals in long reaches of CI and G pipes.

BIOLOGICAL STABILITY

Biological stability for the PDSs are accessed by monitoring of physical and chemical water quality and specific biological parameters. HPCs, AOC, and PEPA (biofilm) are primary biological parameters. Residual, pipe material, and DO are primary chemical and physical parameters.

The effect of residual on effluent HPCs is shown in Figure 10. The resulting relationship is similar to a power equation and shows that HPC growth explodes at very low residuals. Hence, the importance of maintaining a 0.6 mg/L combined residual is illustrated here.

Biostability was defined by regions, as illustrated in Table 4. In Regions I–III, residuals were lower than 0.6 mg/L and HPCs in the effluent were greater than 10^4 cfu/mL; at residuals near zero, HPCs of 10^5 to 10^6 occurred. Waters in these regions had elevated HPCs even when

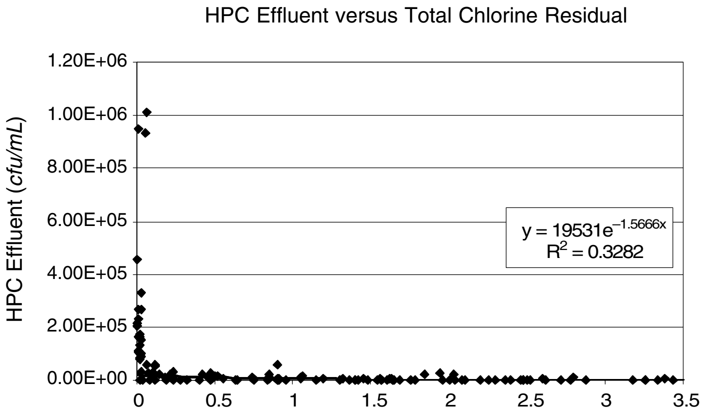


Figure 10 Effect of chloramine residual on HPCs

Table 4 Biostability by region

Regions I–III Residual 0.6 mg/L HPC_{eff} of 10^4 cfu/mL frequent; approaching zero residual 10^5 dominant, and $>10^6$ occurs	Regions I–III Most waters will have elevated HPCs
Region IV Residual 0.6 mg/L AOC of 2.0 mg/L significant	Region IV AOC instability is what differentiates low-HPC ($<10^4$) from high-HPC ($>10^4$) waters
Region V Residual > 2.0 mg/L $HPC_{eff} < 10^4$ #/mL	Region V Most waters will have low HPCs (except high AOC?)

AOC levels were low. In Region IV, residuals were 0.6 to 2.0 mg/L, AOC was significant, and differentiated effluent HPCs were above or below 10^4 cfu/mL. Region V was defined by residuals greater than 2 mg/L and effluent HPCs always lower than 10^4 cfu/mL and typically lower than 10^3 cfu/mL. Most waters had low effluent HPCs in this region and stable AOCs. However, there was no opportunity to evaluate finished waters with AOCs greater than 109 μg/L in Region V, so it may be that higher AOC waters in Region V would have HPC proliferation (and thus exhibit behavior more like that seen in Region IV). However, this is speculative.

In Region IV, one of the major observations is that there were no HPC counts below about 10^4 cfu/mL when AOC changed by more than 37 μg/L (i.e., AOC was consumed, or there was an increase in AOC due to fermentation and hydrolysis of biodegradable organic material). Biostability was achieved in Regions IV and V as defined by effluent HPCs < 10^4 cfu/mL, and often less than 10^3 cfu/mL (note: PDS counts were higher than typical full-scale mains perhaps because they simulated a long HRT condition, 5 days). Using this definition of biostability ($<10^4$ cfu/mL), biologically stable water was observed when influent AOC was less than 109 μg/L, the absolute change of AOC was less than 37 μg/L, and the residual was never less than 0.6 mg/L.

The relationship of the HPCs to material and residual is shown in Figure 11. Clearly residual was affected by material and HPCs increased as residual decreased. The unlined metal pipes had low residuals and high HPCs, clearly promoting biologically unstable water.

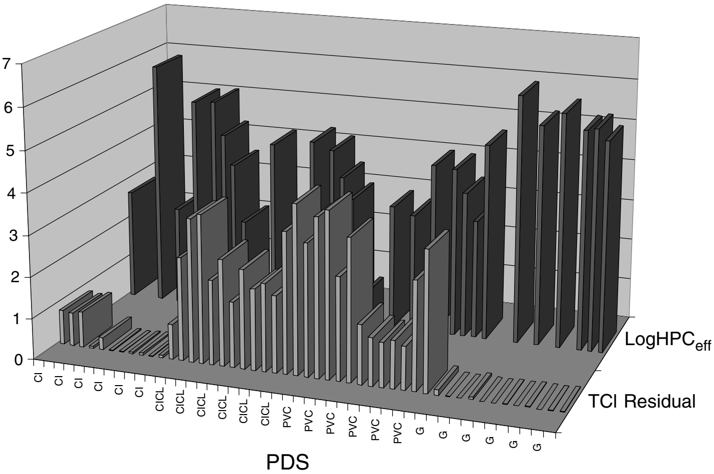


Figure 11 Relationship of HPCs to material and residual

The relationship of biofilms to material and residual is shown in Figure 12. Unlined metal pipes resulted in low residual, but biofilms did not proliferate at low residuals as bulk liquid HPCs did.

The relationship between blend and the absolute AOC change, effluent HPCs, and biofilms (PVC) is shown in Figure 13. RO had most

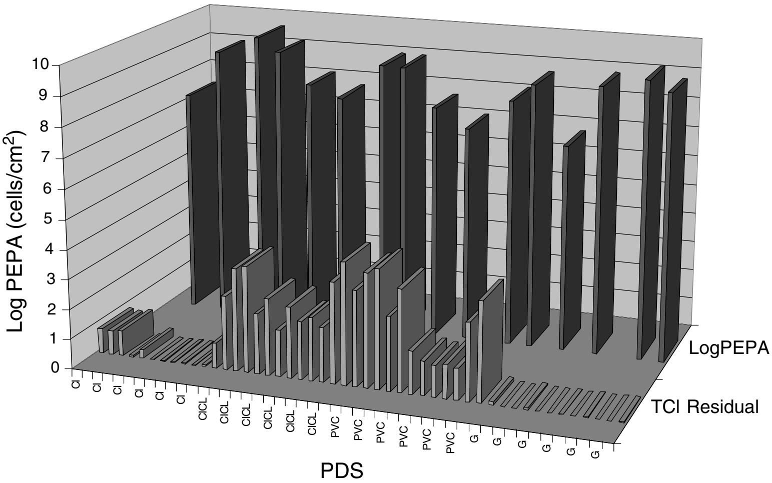


Figure 12 Relationship of biofilms to material and residual

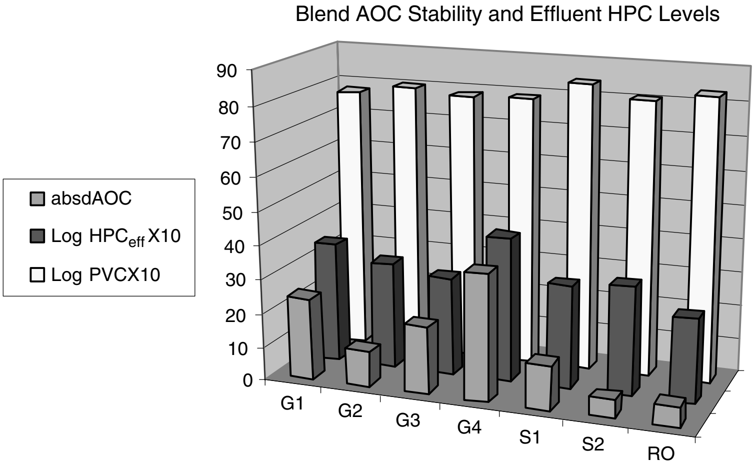


Figure 13 Absolute AOC change, biofilm (PVC), and effluent HPCs versus blend

stable AOC and lowest HPC and was the most biologically stable water. G4 had least stable AOC, highest HPC, and lowest biofilm, and was the least biologically stable water. The biological stability of the remaining waters varied from that of the RO and conventionally treated groundwater.

AOC instability correlated with high influent AOC, not residual. Influent AOC is a good predictor of AOC instability, but high influent AOC does not always predict biological instability. Sometimes low-influent AOCs led to AOC increases in the PDS, presumably from BOM fermentation or hydrolysis to AOC.

A biocide-to-food ratio above 22.9 was consistent with AOC stability (<37 μg/L).

In general, groundwaters tended to be less biologically stable with respect to the bulk liquid but had lower biofilms. RO was the most biologically stable with respect to the bulk liquid but had high biofilms. This suggests the possibility that systems with high biofilms sloughed less material and had a lower steady-state biofilm growth rate, but a higher steady-state biofilm inventory.

HPC was most affected by residual and AOC stability, while biofilms were affected by temperature and material. HPC proliferated above 10^4 cfu/mL at residuals below 0.6 mg/L, rising sharply as zero residual was approached.

HPC proliferation correlated with residual and AOC (especially AOC changes). DO depletion was also found to correlate well with HPC proliferation and was found to be a good indicator of bioinstability. RO produced the most biologically stable water in terms of HPC and AOC levels. NF did not give the same benefit.

Biofilms increased with temperature and on unlined metals. HPCs did not correlate consistently with biofilm levels. Biofilm trends were often inversely related to bulk liquid HPC trends *when comparing blends* (e.g., many blends leading to high biofilms led to low HPCs). Low biofilms coincided with low bulk liquid HPCs *when comparing materials* (e.g., PVC had both low biofilms and low HPCs/high residuals). In the latter comparison, since the materials resulting in low biofilms and low HPC also resulted in high residuals, the HPC trend was probably more a function of residual concentrations than a material effect per se.

Average combined residuals should equal or exceed 0.6 mg Cl_2/L combined chlorine, and the best biological quality was observed at a residual of 2.0 mg/L or more. Maintenance of AOC stability (ABSdAOC) < 37 µg/L and influent AOCs < 109 µg/L are desirable for biostability. A biocide-to-food ratio > 22.9 is recommended when AOC instability is observed.

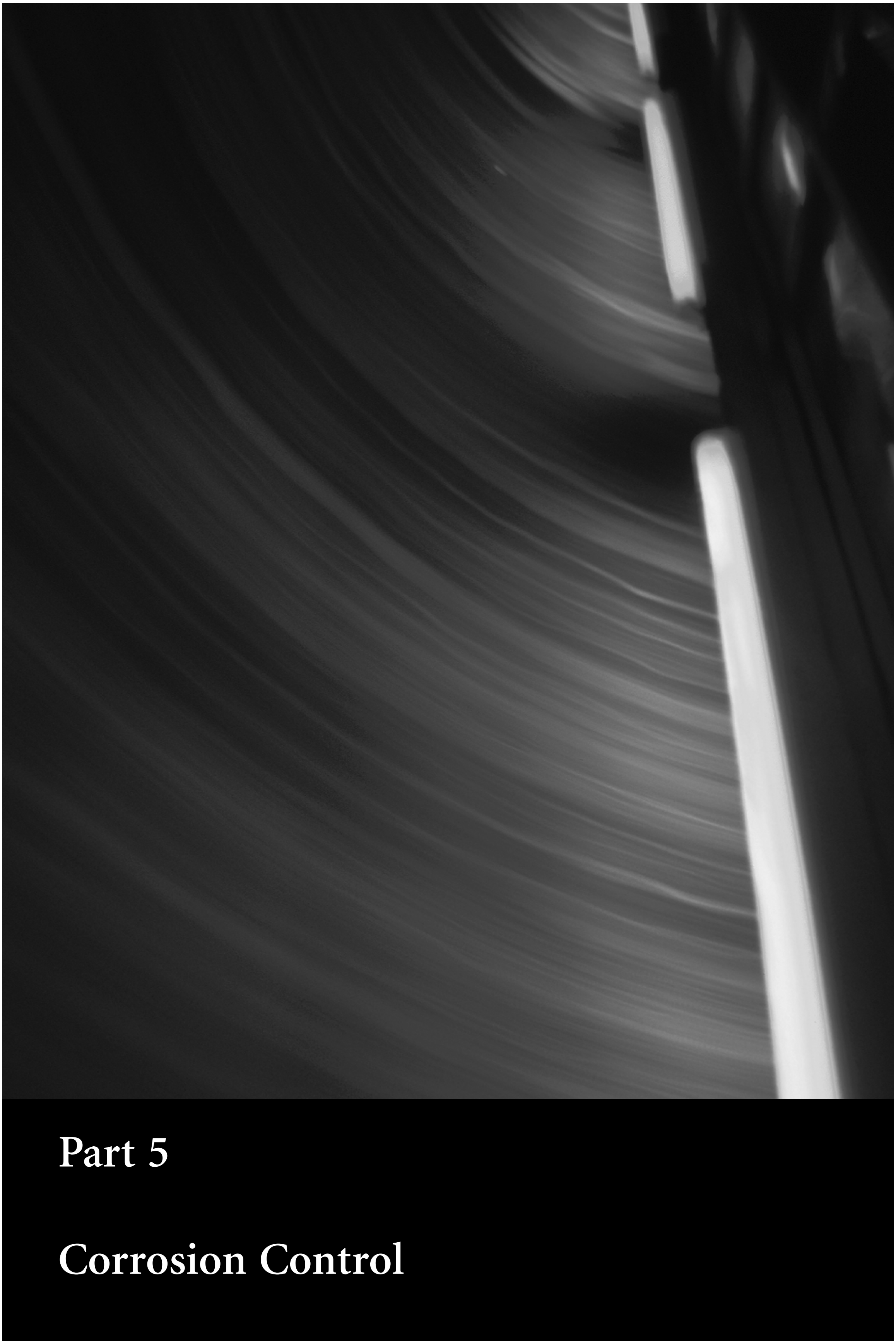
CONCLUSIONS

- Apparent Color
    - Apparent color release from unlined iron-based pipes occurred when alkalinity was lower than background alkalinity of the historical groundwater source. Chloride, sulfate, sodium, and temperature are also indicated as significant variables for color release.
    - The release of apparent color was modeled using a zero-order kinetic model that related release to Reynolds number and water quality.
- Iron, Copper, and Lead Release
    - Iron dissolution was most accurately modeled assuming siderite was the controlling solid phase. The predicted iron concentrations were higher than the dissolved iron concentrations and lower than the total iron concentrations, indicating particulate release of iron.
    - The equilibrium models based on the solubility of tenorite and cupric hydroxide underestimated copper levels due to unaccounted-for particulate copper releases.

  - Empirical modeling of water quality and copper concentration predicted that copper will increase with decreasing pH and increasing alkalinity, which was in agreement with thermodynamic models.
  - The action level for copper was predicted to be exceeded when the high-alkalinity groundwater is in use.
  - Particulate lead is the major form of lead.
  - Empirical modeling of water quality and lead concentration indicated that the action level for lead will not be exceeded.
- Residuals
  - Models for chlorine and chloramine residual were developed as a function of material and water quality.
  - Unlined metal-based pipes exhibited significantly greater residual demand than did lined pipes.
- Biostability
  - Biostability was described by regions relating HPCs, residuals, and AOC.
  - The most biologically stable water corresponded to residual greater than 2 mg/L, and changes in AOC of less than 37 μg/L.
  - RO-produced water was the most biologically stable.
  - Conventionally produced groundwater was the most biologically unstable.

ACKNOWLEDGMENTS

This research was supported by Tampa Bay Water and AwwaRF. The authors are gratefully indebted to AwwaRF project officer Roy Martinez, TBW, and the TBW member governments.



Part 5

Corrosion Control

CHAPTER · 18

Effect of Phosphate Inhibitors on Lead Release From Pipes

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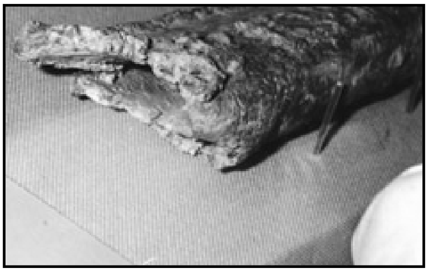
SUMMARY

Hexametaphosphate tends to increase release of both particulate and soluble lead to drinking water. In this study, for every milligram per liter of residual hexametaphosphate, soluble lead increased by ~1.6 mg/L after 72-hr stagnation in pure lead pipes compared with the same condition dosed with orthophosphate. Utilities need to consider these adverse effects whenever polyphosphate is used to prevent scaling or iron precipitation; in fact, polyphosphates cannot be recommended for lead corrosion control. Although soluble lead release in these experiments was in reasonable agreement with predictions of solubility models, particulate lead release was dominant. In some cases, a peak in lead release was observed as water was held stagnant in pipes. Analysis of these data indicated that particulate lead can reattach to the pipe surface under some circumstances.

INTRODUCTION

The beneficial use of phosphates in preventing calcium scale buildup and iron precipitation in water distribution systems has been established for more than 50 years (Rice & Hatch, 1939). It is also known that under some circumstances, phosphates can prevent certain aspects of iron pipe corrosion including red water, weight loss, and tubercle buildup (McNeill & Edwards, 2000; Boffardi, 1988; Rice & Hatch, 1939). Consequently, when the US Environmental Protection Agency Lead and Copper Rule effectively expanded the meaning of the term "corrosion control" to explicitly consider lead and copper concentrations at the consumer's tap,



Lead pipe is well-known for its outstanding corrosion resistance and plumbing properties, as shown by this example of Roman pipe displayed in The British Museum.

phosphate dosing held the promise of providing a low-cost approach to meeting multiple water quality objectives (Boffardi, 1988).

Phosphates can be dosed as either phosphoric acid, combinations of orthophosphoric acid and zinc orthophosphate, polyphosphates, or blends of orthophosphoric acid and polyphosphate. In a 1992 survey of US utilities, more than half of those participating reported that control of lead corrosion by-products was an important reason for their use of phosphate-based corrosion inhibitors (Dodrill & Edwards, 1995; Dodrill, 1992). A 1994 follow-up survey suggested that phosphate use had increased markedly as water utilities gradually complied with the provisions of the Lead and Copper Rule (Dodrill, 1992). About one third of utilities dosing with phosphate inhibitors used either orthophosphate or zinc orthophosphate; the remainder used either pure polyphosphate or a blend of polyphosphate and orthophosphate.

The type of phosphate matters because simplistic solubility models (Figure 1) predict that orthophosphates and polyphosphates might have opposing effects (Schock et al., 1996; Holm & Schock, 1991a). That is, dosing of orthophosphate might be predicted to decrease lead solubility through formation of relatively insoluble scales. In contrast, dosing of polyphosphate inhibitors might be expected to increase lead solubility—previous research clearly established the strength of the lead polyphosphate complex and the probability that it would increase lead leaching to potable water. To the extent that soluble metals are responsible for higher lead and copper at the consumer's tap, dosing of polyphosphates could increase consumer exposure to lead and copper in drinking water. Although this hypothesis is controversial and has been debated for at least a decade (Boffardi, 1991; Holm & Schock, 1991b), recent monitoring data suggest that these concerns have merit (Cantor et al., 2000; Dodrill

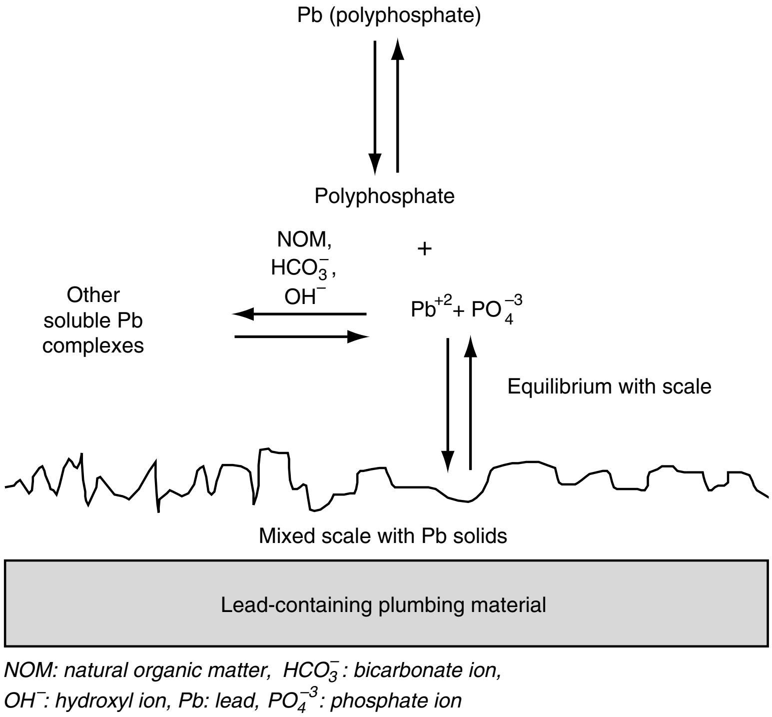


Figure 1 Conceptualization of factors influencing soluble Pb concentrations at equilibrium after stagnation in waters dosed with hexametaphosphate and orthophosphate

& Edwards, 1995), even though the experimental designs of earlier work did not always support drawing of explicit cause-and-effect relationships. Previous research into the effects of polyphosphates is unavoidably complicated by many factors, including reversion of polyphosphates to orthophosphates in water distribution systems (Zinder et al., 1984). Water sampled at consumers' homes invariably contains significant concentrations of orthophosphate, even if only polyphosphate is dosed at the treatment plant (Edwards et al., 2000; Goldberg, 1998). Under these circumstances, it is impossible to attribute potential benefits or detriments exclusively to orthophosphate or to polyphosphate because the net effect involves an interaction between the two.

The work discussed here was designed to provide some unambiguous answers about phosphate's effectiveness in controlling soluble lead release in representative tap waters. Pipe age (or exposure duration) was a key consideration in this analysis, and the authors took a unique approach to determine the relative effects of inhibitors in pipes after a few weeks, a few months, and a few years of aging. They also attempted to sort out the relative effect of polyphosphates versus orthophosphates.

EXPERIMENTAL METHODS

Pipe rig tests (Lind-Johansson, 1989) were conducted to demonstrate changes in lead corrosion by-product release in response to water quality changes and aging. The rigs were 12 in. (305 mm) long, 0.75 in. (20 mm) in diameter, pure lead pipes with a volume of approximately 100 mL. Rubber stoppers were used to plug the ends of each tube. All pipes were initially rinsed three times with 0.1 *N* sodium hydroxide (NaOH) to remove possible organic deposits and then were rinsed five times with reagent-grade purified water. Boulder, Colo., tap water was used as a base water (Table 1), with appropriate modifications using carbon dioxide gas, perchloric acid, or NaOH to achieve other targeted pH and alkalinity conditions (Table 2). The pH and alkalinity conditions identified by

Table 1 Typical characteristics of base water

Parameter	Value
Orthophosphate (mg/L)	< 0.05
Iron (mg/L)	1.1
Lead (mg/L)	< 0.003
Copper (mg/L)	< 0.01
Natural silica (mg/L)	5–8
Calcium (mg/L)	21
Magnesium (mg/L)	7.6
Chloride (mg/L)	6
Fluoride (mg/L)	1.1
Specific conductance (µmhos/cm)	93
Sulfate (mg/L)	6
Free chlorine (mg/L)	0
Total organic carbon (mg/L)	1.28

Dodrill and Edwards were found in a national utility survey to be of "highest interest" with respect to phosphate inhibitor use (Dodrill & Edwards, 1995).

For each of the five water conditions shown in Table 2, the authors tested three phosphate conditions (no inhibitor, orthophosphate, and hexametaphosphate) for a total of 15 water conditions. This allowed direct and unambiguous comparison of inhibitor effects in a given water.

Table 2 pH and alkalinity modifications to Boulder, Colo., water for pipe experiments

pH	Carbonate Alkalinity, *mg/L as $CaCO_3$**	Adjustment to Tap Water	Reason for Interest
7.8	45	No major adjustment required	No effect or adverse effects for both Pb and Cu when polyphosphate inhibitors were dosed.
7.2	300	Sodium bicarbonate added; water bubbled with CO_2	Utilities had a high likelihood of exceeding the Cu action level, and a better understanding of resultant inhibitor effects on Pb release was needed.
9.5	45†	Sodium hydroxide added	Inhibitor effects on Pb release were highly variable.
7.2	15	Perchloric acid added; water bubbled with air	Utilities with this approximate water quality have the highest likelihood of exceeding the Pb or Cu action level. All phosphate inhibitors effectively mitigated Pb and Cu corrosion by-products.
7.2	45	Water bubbled with CO_2	Phosphate inhibitors had beneficial effects for Cu but had no effect or adverse effects on Pb corrosion by-products.

* $CaCO_3$: calcium carbonate, CO_2: carbon dioxide, Pb: lead, Cu: copper

† 60 mg/L total alkalinity; all conditions at 45 mg/L alkalinity were also constant with a dissolved inorganic carbon concentration of 11 mg/L.

Stock solutions of orthophosphate and polyphosphate inhibitors were prepared at concentrations of 500 mg/L and 320 mg/L as P, respectively, and were kept in the dark at 4°C. Standards were typically made up fresh weekly. The polyphosphate standard was made from sodium metaphosphate;[*] when this chemical was dosed to pure water, the phosphorus was 8% orthophosphate even at time zero. For experiments with inhibitors, phosphates were dosed at a concentration of 1.0 ± 0.05 mg/L as P. The background concentration of total phosphate in Boulder tap water was always < 0.03 mg/L as P.

To test the effects of aging or pipe exposure time on lead corrosion by-product release, one batch of pipes was exposed to water beginning in June 1994, and other batches were exposed beginning in January 1997 and June 1997. During the 72-hr sampling event described in this work, these pipes had experienced three years, six months, and two weeks of exposure and in the discussion that follows are termed old, new, and brand new, respectively. These pipes were two weeks older during the 8-hr sampling event described later in this chapter. Thus, a total of 45 lead pipes were tested in these experiments (three pipe ages multiplied by three inhibitor conditions multiplied by five water conditions). Although the lead pipe used in aging experiments was made by a single manufacturer, the brand-new pipe was not produced in the exact same batch as the old and new pipe and was 50% larger in diameter. This factor was considered in subsequent analyses.

The pipes were maintained by regular water changes every Monday, Wednesday, and Friday for the duration of the study. During the water changes, the pipe was first inverted three times to suspend settled particles and then all of the water was poured out. The pipe was immediately refilled with fresh solution of the appropriate water quality. Pipes were laid flat, and water inside the pipes was stagnant between water changes. It is impossible to know whether this protocol would increase or decrease particulate lead release in comparison with a pipe in a home.

On occasion, pipes were intensively sampled to determine characteristics of lead by-product release to the water. Replicate samples were collected from the pipes for three successive 72-hr stagnation periods (e.g., Sunday, Wednesday, and Saturday) so that the mean, standard deviation, and 95th percentile confidence interval of release from each pipe could be calculated. Compared with actual exposure in distribution

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[*] Fisher Scientific, Houston, Texas

systems, this protocol has some obvious limitations in terms of stagnation time, pressures, and shear forces; however, its advantage over other, more complicated flow-through apparatus was that it provided absolute control over water quality—the key focus of this research.

Lead and total phosphorus were analyzed by an inductively coupled plasma emission spectrophotometer[*] according to method 3500 or 4500, respectively (*Standard Methods*, 1998). Total lead was determined after samples had been acidified with 1% concentrated nitric acid, whereas soluble lead was operationally defined by filtration through a 0.45-µm-pore-size syringe filter. Because colloidal species can sometimes pass through this filter, the filtration approach represents an upper bound to truly soluble lead. Orthophosphate was distinguished from polyphosphate using a colorimetric test with ascorbic acid as described in method 4500-PE. Quality assurance and quality control, in which soluble lead and phosphate spikes were recovered after filtration and storage, established that filters and bottles were not sorbing these species (Edwards et al., 2000).

RESULTS AND DISCUSSION

In this work, the authors took two approaches to examine the effects of time on by-product release phenomena. First, by-product release was tracked continuously from a given pipe in a particular water quality for a period as long as four years. Second, because this study used an actual tap water with inherent typical seasonal and other variations in water quality, another experiment was conducted in parallel in which pipes had been exposed for a few months or a few weeks. By measuring by-product release from all these pipes on the same day using the same water, effects of aging could be directly determined. At various points in the sampling, particulate and soluble metals were determined, along with measurement of total phosphate residual and orthophosphate.

The potential limitations of the work must be noted from the outset. First, the trends for inhibitors observed in this study may not be directly transferable to other waters of similar pH and alkalinity. Although pH and alkalinity are believed to be two of the most important factors influencing corrosion and inhibitor behavior, other factors (e.g., chloride, hardness, disinfectant type and dose, stagnation and flow, and

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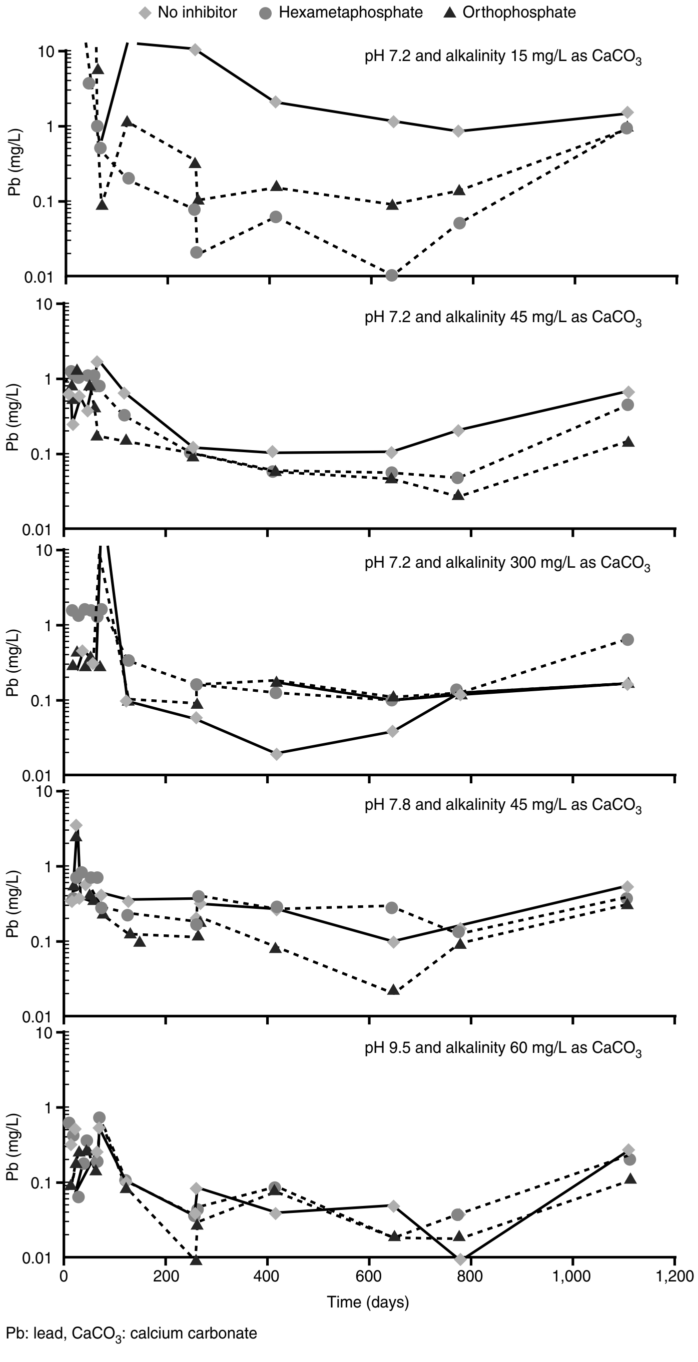
[*] Jobin Yvon, Ultima, Edison, N.J.

temperature) are undoubtedly important in at least some circumstances. Second, with respect to lead release, it is well-known that mechanisms of release differ based on the source of lead (i.e., brass, pure lead, lead-tin solder), whether the material is galvanically coupled to a more noble metal such as copper, and other factors. This work examined the effects of water quality on corrosion of pure lead pipes exclusively. Given the lack of information regarding corrosion of other lead-containing plumbing materials, it is often assumed that trends noted for pure lead apply. Although this assumption may provide some basis for decision making, the validity of this hypothesis has not been rigorously tested, and in fact, there is reason to believe that it is not valid in many circumstances. Thus, as with any research, the results presented here must be applied cautiously to other situations.

Effects of Aging, Inhibitors, and Stagnation Time on Total Lead Release

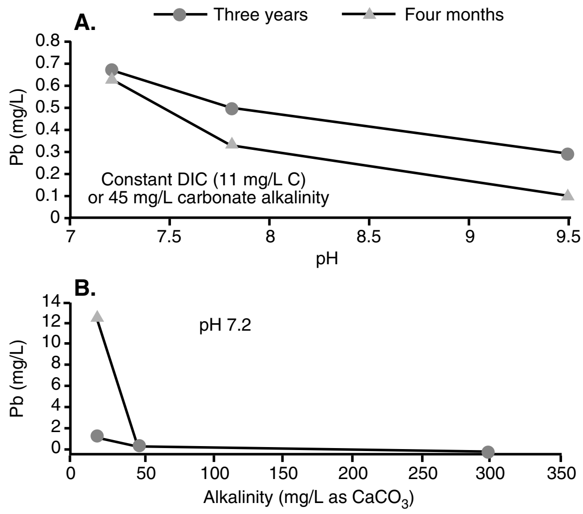
One perspective on the role of aging and water quality in lead by-product release can be obtained by tracking a single pipe over a long period of consistent exposure. Because all the experimental solutions in this research started with “real” tap water, factors affecting release from these pipes include aging, seasonal effects, and subtle changes in water quality parameters other than pH and alkalinity (e.g., total organic carbon, sulfate, chloride, and other ions). In the discussion that follows, the term “pH” refers to the initial pH of the water added to the pipe, except when it is specifically noted as the pH after stagnation.

For new lead pipes without inhibitors at pH 7.2, by-product release was much lower at 45 and 300 mg/L alkalinity than at 15 mg/L alkalinity (Figure 2). The benefits of higher alkalinity at this pH were nearly all accrued in the change from 15 to 45 mg/L. With aging, by-product release dropped by about an order of magnitude at pH 7.2 at four months versus three years at alkalinity of 15 mg/L as calcium carbonate ($CaCO_3$). At the higher alkalinity, however, aging did not appear to decrease by-product release that much over the same time span. Nevertheless, at any given age, the benefits of 45 versus 15 mg/L alkalinity were still very substantial. When pipes at constant dissolved inorganic carbon and varying pH were examined (Figure 3), lead release in the absence of inhibitors was also benefited by higher pH. The effect tended to be relatively linear with pH regardless of pipe age, in contrast to the apparent threshold effect of alkalinity. Lead release lessened as the pipe aged from four months to three years, with more significant percentage improvements from aging at the higher pH.



Pb: lead, $CaCO_3$: calcium carbonate

Figure 2 Total Pb release from a pipe after a 72-hr stagnation period and indicated experimental time

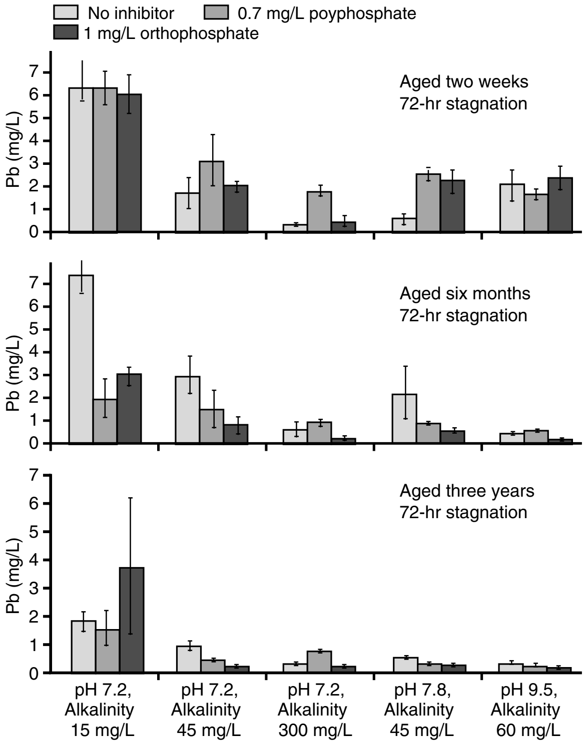


$CaCO_3$: calcium carbonate, DIC: dissolved inorganic carbon, Pb: lead

Figure 3 Effects of aging on Pb release at constant DIC concentration (A) or constant pH (B)

At most pipe ages, the benefits of inhibitors were noteworthy, even on a logarithmic scale at pH 7.2 and 15 or 45 mg/L as $CaCO_3$. The most obvious exception to this occurred at pH 7.2, alkalinity of 300 mg/L; under these conditions, polyphosphate clearly increased lead release compared with the system with orthophosphate or no inhibitor in relatively new pipes. As a general rule, by-product release decreased about 0.5–1 order of magnitude with aging over the three-year experiment; however, a much more substantial improvement was noted for pipes at pH 7.2, alkalinity of 15 mg/L as $CaCO_3$. The lowest lead releases were noted at pH 9.5 with or without inhibitor dosing, which is consistent with predictions based on solubility models and utility monitoring data (Edwards et al., 1999; Schock, 1989; Schock & Gardels, 1983; Schock, 1980). However, very low levels of release were also obtained at pH 7.2 and 300 mg/L as $CaCO_3$ without inhibitor.

For another perspective on aging, the authors looked into whether these trends could be confirmed with statistical confidence for the pipes of different ages but exposed to the exact same water on the same day. In pipes without inhibitors, the trend of lower lead release with higher alkalinity at pH 7.2 was confirmed at > 95% confidence for all pipe ages (Figure 4). This was also the case when the test was repeated with an 8-hr



Error bars indicate 95% confidence interval based on triplicate samples. Pb: lead

Figure 4 Total Pb released during 72-hr exposure

stagnation, except that a high variability in lead release at pH 7.2 and alkalinity of 15 mg/L for the three-year-old pipe decreased the significance of the trend to < 95% confidence (Figure 5).

Generally, lead release significantly decreased with aging, although an unusual increase in release was noted in six-month-old pipes at pH 7.2 and 7.8 without inhibitors, compared with the same condition for pipes two weeks in age. This might be at least partly attributable to the 50% larger diameter of the pipes aged just two weeks (noted previously); such an interpretation is consistent with the fact that increased lead release was not observed during this period for data in Figure 2.

The relative benefits of inhibitors were assessed by using the following equation:

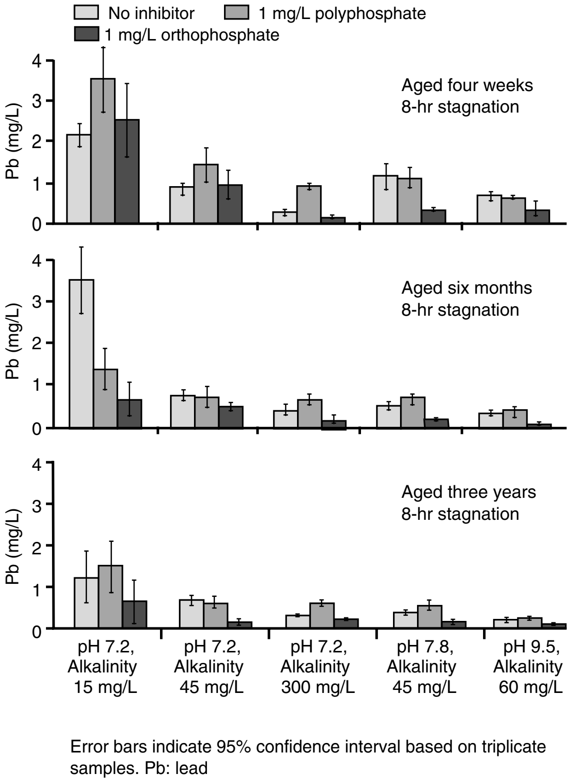


Figure 5 Total Pb released during 8-hr exposure

$$\text{Percent change} = \frac{(\text{Pb release with inhibitor} - \text{Pb release without inhibitor}) \times 100\%}{\text{Pb release without inhibitor}} \qquad (1)$$

In this calculation, benefits attributable to inhibitor dosing resulted in negative numbers, whereas detrimental effects resulted in positive numbers. In the discussion that follows, only "significant" effects of > 95% confidence are presented, although trends of lesser confidence are clearly apparent in Figures 4 and 5. A broad compilation of inhibitor effects for the two stagnation times, differing pipe ages, and all water qualities is instructive (Figure 6). Orthophosphate dosing often led to > 70% decreases in lead release. This was true for both the 8- and 72-hr

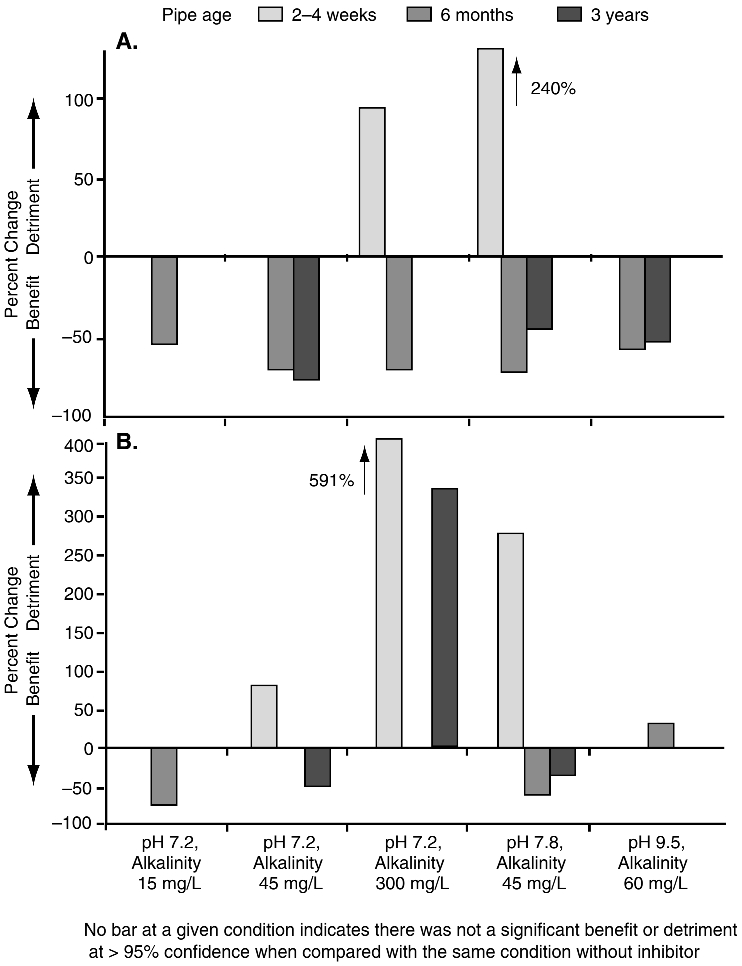


Figure 6 Relative effect of orthophosphate (A) and polyphosphate (B) on total lead released for 72-hr triplicate sampling

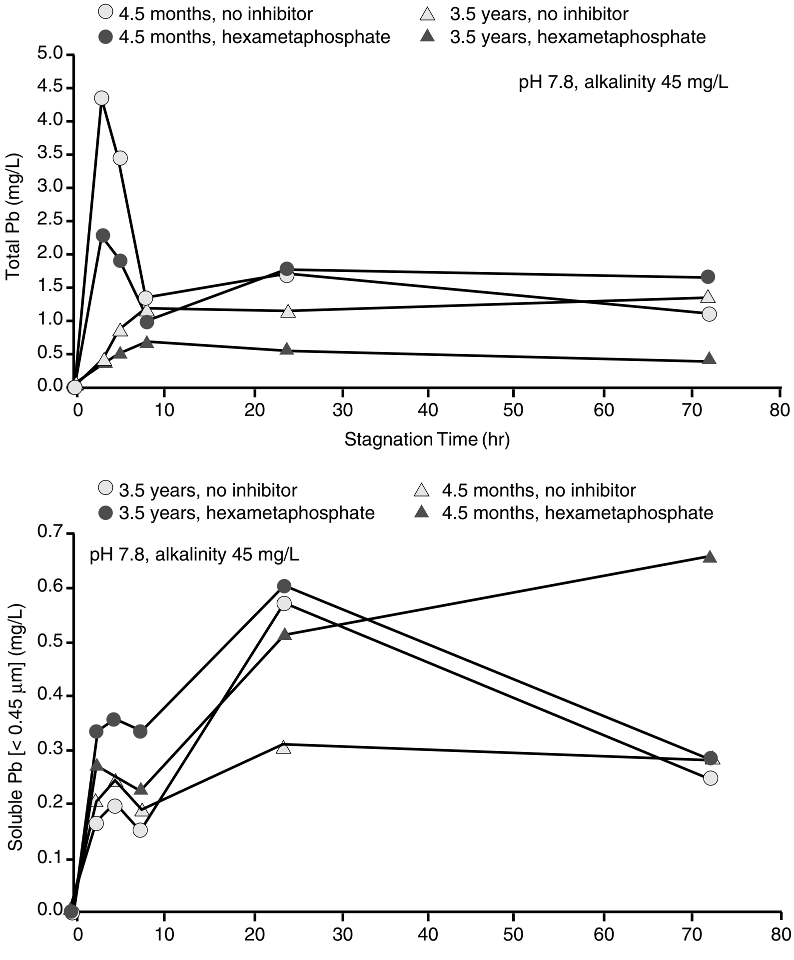
stagnation times at every water quality tested at six months' pipe age. In pipes aged three years, orthophosphate significantly (> 95% confidence) reduced lead release for all conditions except pH 7.2, 15 mg/L alkalinity. The only detrimental effects from dosing orthophosphate were in brand-new pipes (aged two weeks) with a 72-hr stagnation time in waters at pH 7.2, 300 mg/L alkalinity, and pH 7.8, 45 mg/L alkalinity.

In contrast, the effects of hexametaphosphate dosing were more frequently detrimental than beneficial, compared with the same condition without inhibitor. In fact, during the 8-hr stagnation sampling, the only benefit was at pH 7.2, 15 mg/L alkalinity at six months' pipe age. In the brand-new pipes (two weeks old) and the old pipes (three years old), hexametaphosphate was highly detrimental to lead release at pH 7.2, 300 mg/L alkalinity for both an 8- and a 72-hr stagnation time, with as much as a 591% increase in lead release. This confirmed trends noted earlier in this water when lead release from a given pipe was tracked over a three-year time span (Figure 2). However, pH 7.2, alkalinity 300 mg/L as $CaCO_3$ was the only condition for which adverse effects from hexametaphosphate were still significant at > 95% confidence after three years.

One of these results was particularly perplexing and deemed worthy of follow-up study. For the condition at pH 7.8, 45 mg/L alkalinity, polyphosphate significantly worsened lead release for the two-week-old samples and improved release in the older samples at a 72-hr stagnation time. In contrast, polyphosphate had nearly no effect at 8-hr stagnation, regardless of pipe age. To better understand how these results could occur, the authors resampled the pipes at stagnation times of 3, 5, 8, 24, and 72 hr. This was not a triplicate analysis, and it must also be noted that the pipe that had been aged two weeks in Figure 6 was more than four months old by the time this experiment was conducted. Nevertheless, as noted previously in the triplicate analysis, by-product release followed some unusual trends with time in these pipes (Figure 7).

For the most part, soluble lead increased with time before leveling off after about 3 hr. In the older pipe samples, total lead release followed the same pattern, although lead release in samples with hexametaphosphate leveled off at much lower values than in the sample without hexametaphosphate. In the newer samples, total lead release appeared to peak after ~5 hr, after which time it dropped sharply before eventually leveling off. This caused the perplexing result that after 72-hr stagnation for the new samples, the system with hexametaphosphate showed higher lead whereas the opposite was true for time periods < 24 hr.

This result is interesting because this type of release, with a peak at 5 hr followed by a decrease thereafter, has previously been reported for copper under anoxic conditions (Edwards & Boulay, 2001; Werner et al., 1994). The other noteworthy finding is that the initial peak in release clearly arose from mostly particulate lead, which seemingly "reattached" to the pipe wall, whereas in the past, reattachment has been attributed to redox changes from a more soluble metal species to a less soluble species [i.e., Cu(II)→Cu(I)]. Finally, this unusual release curve highlights another limitation of this work; conclusions based on the 8- and 72-hr sampling events, which provide important snapshots of by-product



Pb: lead

Figure 7 Pb released with time from pipe aged 3.5 years or 4.5 months at pH 7.8, 45 mg/L alkalinity

release trends in time, cannot always be extrapolated with confidence to other stagnation times.

Changes in Soluble Phosphorus During Stagnation

The previous sections described trends in total lead release, which represent the combined result of particulate and soluble lead. This section takes a closer look at chemical changes taking place in pipes during stagnation events, as well as factors influencing the concentration of soluble lead after stagnation. Changes in particulate lead are discussed in detail elsewhere (Edwards et al., 2000). The chemical changes deemed most important to track in this research included pH, phosphate consumption, and polyphosphate reversion to orthophosphate.

During stagnation, pH tended to increase markedly in the pipes (Figure 8), although the magnitude of the shift tended to decrease with pipe age. Although no clear trends were discerned between pH after stagnation and either soluble or total lead release, it is interesting to note that in most cases the final pH was higher in the pipes without inhibitors. This may help explain why in these experiments, longer aging times were needed before the benefits of orthophosphate became apparent; higher pH values generally reduced lead release, and in pipes exposed for shorter periods, the final pH was often higher in waters without inhibitors.

During the 8-hr stagnation event, some soluble phosphate always remained in the water, although as much as 90% of the phosphate was consumed in some cases (Figure 9). Consumption of orthophosphate was inversely related to alkalinity. That is, nearly no phosphate was consumed in the high-alkalinity water at pH 7.2 after an 8-hr stagnation, whereas the highest consumption was noted for the lowest-alkalinity water at the same pH. This would be expected if a lead carbonate solid was forming preferentially to a lead phosphate solid at higher alkalinity or if there was a higher rate of lead corrosion at higher alkalinity.

In 12 of 15 cases, significantly less hexametaphosphate was consumed than orthophosphate. This was most extreme at pH 7.8, 45 mg/L alkalinity in new pipes, for which no hexametaphosphate was consumed and nearly 85% of the orthophosphate was consumed on some occasions. Tests also demonstrated that a substantial portion of the hexametaphosphate was reverting to orthophosphate during the 72-hr stagnation event and the extent of reversion tended to increase with pipe age (Edwards et al., 2000). At pH 7.2, 45 mg/L alkalinity, as much as 80% of the residual soluble phosphate had been converted to orthophosphate during the 72-hr stagnation, and at least 20% of the soluble phosphorus was present as orthophosphate at the other conditions. Thus, the polyphosphates are more readily converted to orthophosphates in older pipes, with concomitant advantages to lead release (discussed in a subsequent section).

A Closer Look at Soluble Lead Release During Stagnation

Soluble lead concentrations in the pipes were always lower in the presence of orthophosphate than in an equivalent system without inhibitor (Figure 10). One significant observation noted by the authors

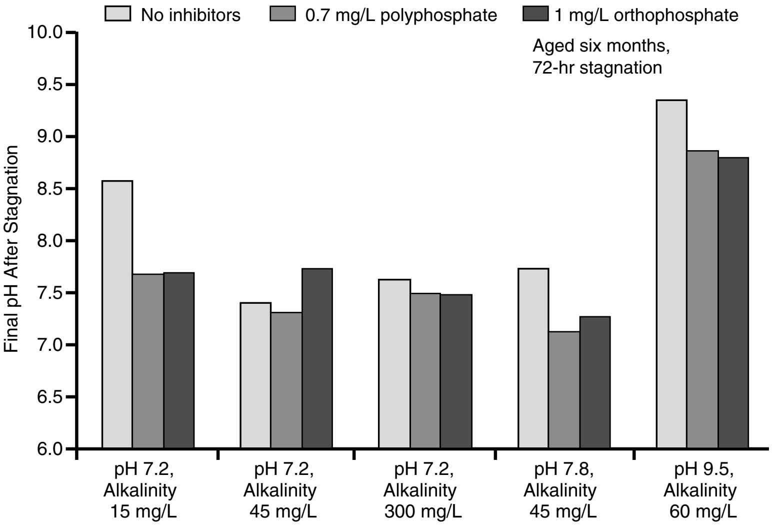


Figure 8 Final pH after 72-hr stagnation for pipes aged six months

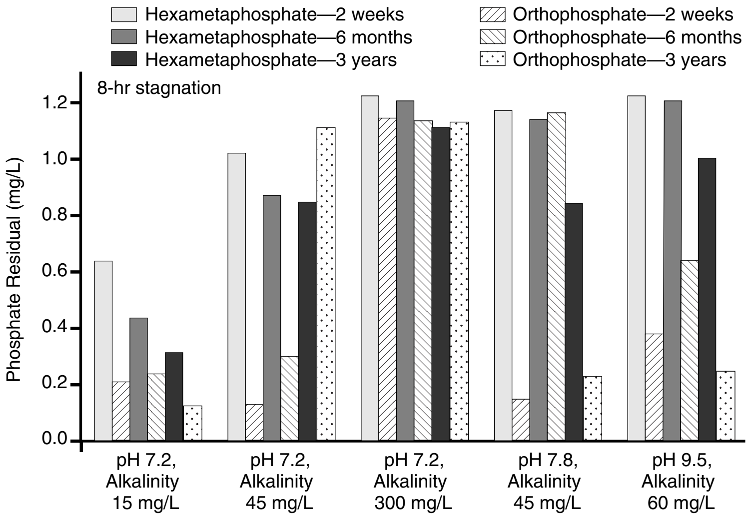
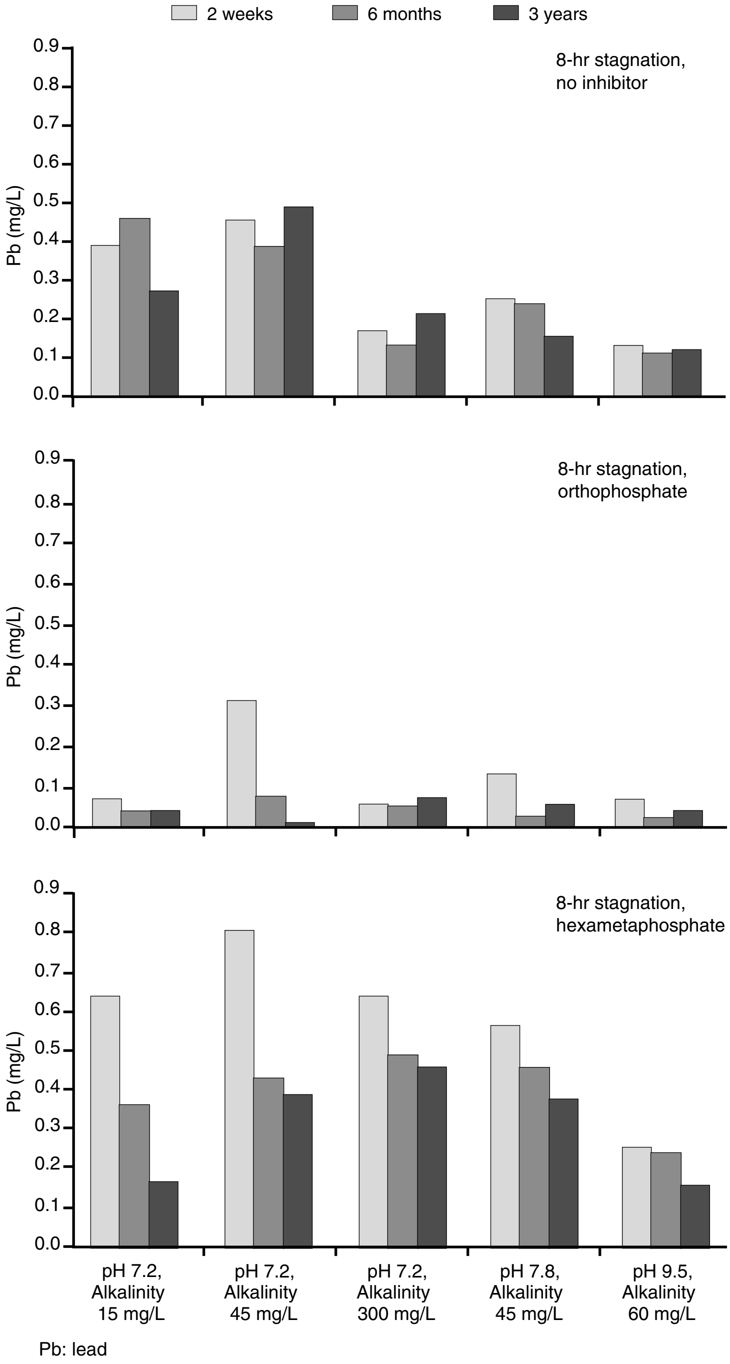


Figure 9 Phosphate residual in pipes dosed with hexametaphosphate and orthophosphate

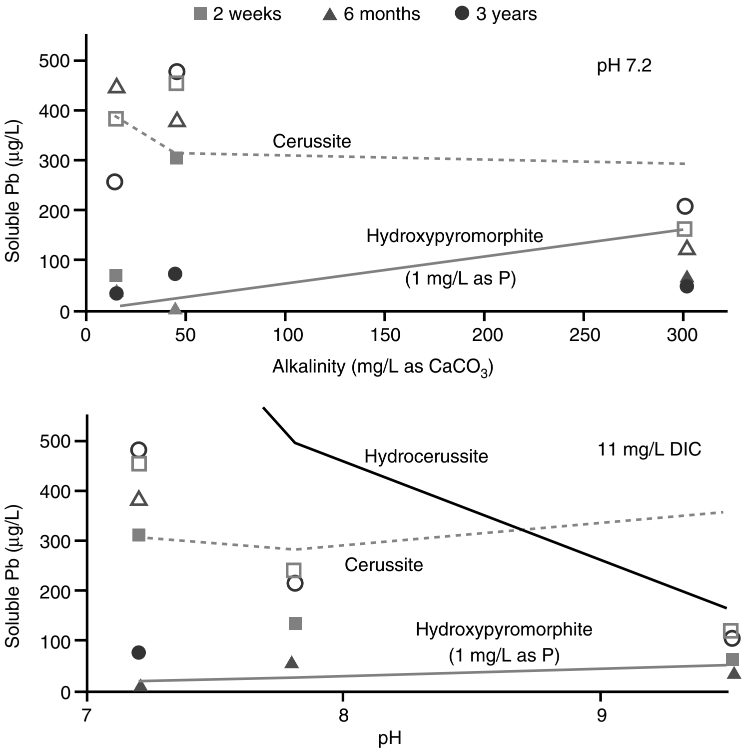
was that in the absence of inhibitors, soluble lead concentrations did not change much with time. Even with inhibitors, when effects of aging on solubility were more significant, decreases in solubility were typically only



Pb: lead

Figure 10 Soluble Pb release in the 8-hr stagnation sampling event about 50% (comparing the brand-new and old pipes). This was in marked contrast to copper pipes, in which solubility is a very strong function of pipe age in some circumstances (Edwards et al., 2001). Of course, in actual practice, pipe ages well beyond the four years tested here are commonplace, and it is quite possible that soluble lead release would decrease substantially under those circumstances.

The actual levels of soluble lead agreed reasonably well with predictions of cerussite solubility in the absence of inhibitors (Figure 11). The exception was the data at pH 9.5, which were more consistent with the lower-solubility hydrocerussite, as would be expected based on solubility models (i.e., Figure 11 and discussion in Schock & Gardels, 1983). Likewise, levels of soluble lead in the presence of orthophosphate



$CaCO_3$: calcium carbonate, DIC: dissolved inorganic carbon, Pb: lead
Lines indicate different solid phases, open symbols indicate without phosphate, and filled symbols indicate with phosphate.

Figure 11 Comparison of predicted levels of soluble Pb in equilibrium with different solid phases and measured levels of soluble Pb in experiments with and without phosphate were in the range of 0.02 to 0.3 mg/L, which was expected based on the levels of 0.1 mg/L predicted in the presence of 1 mg/L as P residual (Figure 10 versus 11). It should be noted that the actual levels of residual phosphate were < 1 mg/L in pipes because of consumption of inhibitor (discussed in a subsequent section); this likely accounts for the lead concentrations > 0.1 mg/L observed in the pipes. In addition, particulate lead was so high in these samples that it is quite likely that a substantial portion of the lead that was defined as "soluble" in Figure 9 was actually colloidal. In any case, soluble lead levels were neither unusually high nor low compared with expectations outlined in prior research (Schock et al., 1996; Schock, 1989; Schock & Gardels, 1983; Schock, 1980).

In every instance, soluble lead concentrations were lower in the presence of orthophosphate than in an equivalent system without inhibitor (Figure 10). Conversely, with very few exceptions, soluble lead concentrations were higher in systems dosed with hexametaphosphate than in those without inhibitor. Therefore, in terms of relative performance of the two inhibitors in controlling soluble lead release, orthophosphate has an enormous advantage. To highlight this effect, the percentage increase in soluble lead attributable to hexametaphosphate dosing compared with orthophosphate can be calculated as

$$\text{Percent increase} = \frac{(\text{Pb release with hexameta-P} - \text{Pb release with ortho-P}) \times 100\%}{\text{Pb release with ortho-P}} \qquad (2)$$

This demonstrates that hexametaphosphate increased lead release in every instance compared with an equivalent dose of orthophosphate (Figure 12). The minimum increase was 100%, or a doubling of the soluble lead concentration. The most pronounced effect was at pH 7.2, 45 mg/L alkalinity in pipes aged three years; under these conditions, hexametaphosphate increased soluble lead by 3,500%. Clearly, hexametaphosphate substantially increases problems with soluble lead under a wide range of circumstances. Moreover, the magnitude of the observed effects is disturbing.

Extensive attempts were made to model the increase in soluble lead using a simplistic approach that previously proved successful for copper (Edwards et al., 2000). The initial assumption was that polyphosphate complexes were exclusively responsible for the "excess" concentration of soluble lead observed in pipes dosed with hexametaphosphate versus orthophosphate. If so, the term "excess soluble lead" would be defined as

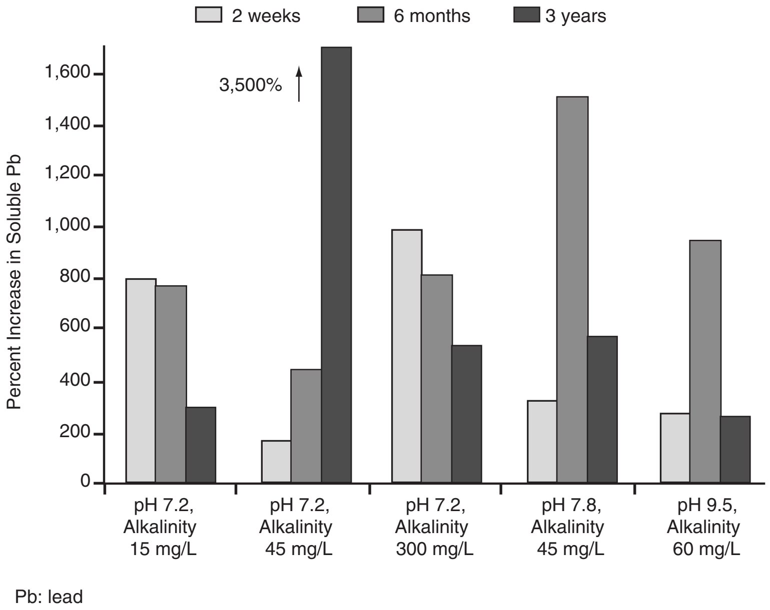


Figure 12 Comparison of percentage increase in soluble Pb attributable to the use of hexametaphosphate versus use of orthophosphate

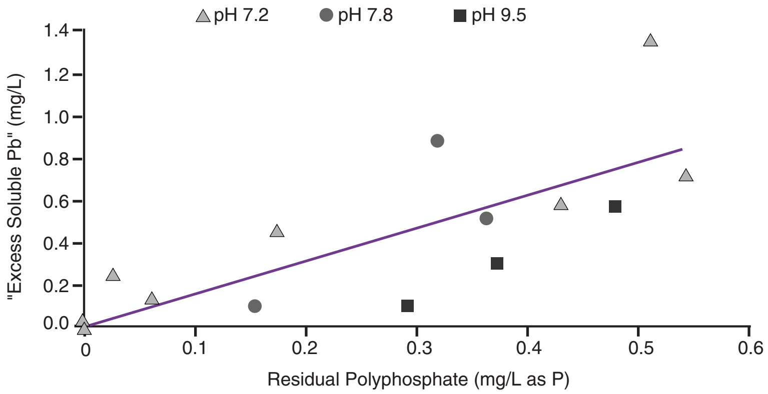
$$\text{Excess soluble lead} = \text{soluble lead}_{\text{polyphosphate}} - \text{soluble lead}_{\text{orthophosphate}} \tag{3}$$

If the lead-polyphosphate complex is very strong, then under some circumstances it can be shown that the following relation will hold:

$$\text{Excess soluble lead} = K\,[\text{total polyphosphate}] \tag{4}$$

in which K is a constant.

There was a reasonable correlation between the "excess soluble lead" and residual polyphosphate at a wide range of pH values and alkalinities, with a best-fit K value of 1.6 (Figure 13). In other words, every 1 mg/L of residual polyphosphate P gives a potential complexation capacity of 1.6 mg/L lead. This observation was in reasonable agreement with the 2 mg/L of lead complexation capacity determined for the hexametaphosphate used in this research (Edwards et al., 2000) and indicates that this



Pb: lead; data from 72-hr stagnation experiment; "excess soluble Pb (mg/L)" = 1.6 residual polyphosphate (mg/L); $R^2 = 0.60$

Figure 13 Excess soluble Pb as a function of residual hexametaphosphate for all pipe ages and water qualities

capacity can be realized under some circumstances. Though data to make clear judgments were sparse, it appeared that lead release at pH 9.5 was distinctly less sensitive to polyphosphate concentration, possibly because of reduced complexation by polyphosphate or calcium interference. This issue deserves additional study.

It should be noted that in many practical situations, the lead complexation capacity would not be realized. For example, in waters with higher hardness than was present in this research (Table 1), a substantial fraction of the polyphosphate that might have complexed lead might be complexed by hardness ions, thereby decreasing the maximum level of lead dissolution (Edwards et al., 2000; Schock et al., 1996). Likewise, if pure lead pipes were not used, it is unlikely that the water would fully equilibrate with lead scale phases (Schock et al., 1996). Given the stringent lead action limit of 15 µg/L, however, realizing only 10% of the theoretical complexation would translate to an additional 20 µg/L lead if residual polyphosphate was 0.1 mg/L P. Thus, the effect could be serious when a drinking water with residual polyphosphate comes into contact with brass, solder, or other lead-bearing materials.

Recommendations to Utilities Dosing or Considering Dosing With Polyphosphates

On the basis of these and other results (Cantor et al., 2000; Dodrill, 1992; Holm & Schock, 1991a), polyphosphate cannot be recommended for lead corrosion control without extensive testing that provides evidence contradicting this research. It is quite possible that under certain circumstances not tested in this study, polyphosphate dosing would decrease lead release. For example, if extensive reversion to orthophosphate occurred before the water came into contact with lead-bearing plumbing materials in consumers' homes, benefits from the produced orthophosphate could outweigh detriments from the remaining polyphosphates. Of course, given that very little is known about reversion kinetics in water distribution systems, it seems risky to rely on reversion reactions to produce the desired benefits.

It is also important to state that polyphosphate still offers important benefits for preventing scaling and precipitation of iron or manganese and facilitating certain types of iron corrosion control. Under circumstances in which polyphosphate is dosed explicitly for these reasons, it would be worthwhile to determine the minimum polyphosphate dose that achieves these desired benefits, because detriments to maximum soluble lead in pipes during stagnation are expected to be proportional to residual polyphosphate (Figure 13). However, ensuring that < 1 µg/L of soluble lead is complexed would require residual polyphosphate of < 0.0006 mg/L.

CONCLUSIONS

Orthophosphate dosing typically reduced soluble lead release by ~70% except in some exceptional cases in new pipes; in these cases, increased lead release was observed compared with the same system without inhibitor. One possible reason for adverse effects of orthophosphate, especially in new pipes, is that pH increased to a lesser extent during stagnation compared with the same condition without inhibitor.

Hexametaphosphate generally increased soluble lead release over a broad range of water qualities. Each milligram per liter of residual hexametaphosphate increased soluble lead release by ~1.6 mg/L when compared with the same system with orthophosphate.

Soluble lead release was in reasonable agreement with predictions of solubility models. Soluble lead concentrations did decrease significantly with aging but to a much lesser extent than particulate lead levels did.

Particulate lead release was the dominant component of the total lead release in this work. In some cases, a peak in lead release was observed during stagnation. Analysis of the data suggested that particulate lead could reattach to the pipe surface during long stagnation times.

ACKNOWLEDGMENTS

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CHAPTER · 19

Testing and Evaluation of Lining Materials for Drinking Water Pipeline Rehabilitation

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ABSTRACT

As the deterioration of aging drinking water distribution systems in North America threatens water quality and system reliability, utilities are exploring new methods of pipeline rehabilitation. In addition to preventing corrosion and improving flow hydraulic characteristics, obtaining any semistructural enhancement of rehabilitated pipes is considered to be beneficial. Although spray linings are traditionally considered to be nonstructural, an innovative attempt is made in this study to evaluate the strength enhancement provided by an epoxy spray lining system. Pressure tests were performed by subjecting 20.3-cm (8-in.) diameter epoxy-lined ductile-iron pipe sections to an internal hydrostatic pressure of 827 kPa (120 psi). The surface of the 0.6-m (2-ft) long pipe sections had circular discontinuities of 0.63 cm (1/4 in.), 1.3 cm (1/2 in.), and 1.9 cm (3/4 in.) diameters to expose the epoxy lining. The results indicated that the epoxy lining sprayed at 2-mm (80-mil) thickness could protect underlying pipes that had corroded areas up to 3/4 in. in diameter, under the test conditions. The lining sprayed at 1-mm (40-mil) thickness conventionally prescribed for holiday-free coating failed at the 3/4-in. diameter exposed areas under the test conditions. The coating at 1-mm thickness would still likely extend service life of the pipe by reducing red water complaints. Full-scale field tests and bench-scale leaching tests in the laboratory were

also conducted as part of this project. Water samples were obtained from customers' taps that were connected to freshly lined water mains. By testing various water quality parameters, water passing through the epoxy-lined mains was determined to be of excellent quality. No leached chemicals were detected in field or laboratory water samples as long as the system was flushed according to standard lining procedure.

INTRODUCTION

The aging infrastructure, particularly drinking water distribution systems, is a well-known problem in many major North American cities. Pipelines form more than 80% of the distribution system and their condition is highly important. Cast iron has been the material of choice in water distribution systems for the last 150 years and now ductile and cast iron together constitute the majority of water main pipelines in the United States [1]. Most central systems of Canadian water and wastewater infrastructure were also started in the middle of the nineteenth century [2]. It is evident that these pipes are nearing the end of their service life and the effects of corrosion further worsen their condition. It is known that cast-iron pipes are susceptible to breaks due to their age, prevalence, and corrosion characteristics. Since the 1980s, the trend in underground pipe installations has been to focus on repair, renovation, and rehabilitation of existing infrastructure as opposed to putting in new service lines [3].

Corrosion is defined as the "physicochemical interaction between a metal and its environment which results in changes in the properties of the metal" [4]. The effects of corrosion include pipe breaks, leakage, reduction in flow due to formation of carbuncles, tuberculation, etc. It must be noted that internal corrosion rarely leads to pipe breaks or failure, primarily due to the formation of protective scales at the surface and subsequent reduction in corrosion rates [5]. Frequently, however, internal corrosion of cast-iron pipes causes the phenomenon of red water. Customer complaints are inevitable, as the condition affects the aesthetic quality of drinking water. Red water is caused by suspended ferric hydroxide flocs produced as a result of iron pipe corrosion. The red water phenomenon is often triggered by changes in flow volume or flow direction caused by high water demand, depressurization after breaks, or closing of valves in the system. Changes in water chemistry, such as pH, alkalinity, chloride, sulphate, or hardness, could also cause an increase in problems.

Many efforts have been undertaken in recent years to combat corrosion in cast iron mains. Some of the methods employed are cathodic protection, external and internal pipeline coatings, changes in water chemistry, and replacement with nonmetallic pipe. The spray lining of organic coatings using trenchless methods has been suggested as a protective barrier against corrosion [6]. The density of activities above the underground pipelines emphasizes the need for trenchless technology. The advantages of this technology include cost savings, reduction in labor, and minimum disruption to the public. In particular, internal lining systems are gaining the interest of public utilities. The rapid application of lining, its curing, and the proven barrier protection are significant advantages of this method. Also, lateral connections are not closed off, allowing faster return to service. Some of the popular rehabilitation spray-lining systems in use today are two-component epoxy resins, cement-mortar, and polyurethane systems. A survey of 53 Canadian municipalities indicates that around 35 of them had used the pipe lining technique at least once [7]. Spray linings are very useful in the repair of pipes with diameters less than man-entry size or between 100 and 900 mm. These are not traditionally considered to provide any structural enhancement to the pipe [3]. Specific liners that are structurally reinforced are said to be effective in providing strength to man-entry-size pipes.

The history of the water distribution system in Edmonton, Alberta, Canada, dates back to 1902 when cast iron was the popular pipe material in use. Currently the distribution system has a total length of about 2,977 km (1,850 miles) [8], with the classification by pipe material as shown in Table 1.

A particular older area of Edmonton had a history of red water complaints stemming from corrosion of cast-iron mains. Although the phenomenon of red water is primarily an aesthetic issue, it is still an indication of underlying conditions. Nondestructive testing of the cast-iron pipes in Edmonton showed that the wall thickness in a corroding pipe can vary along its entire length, since the pipe is exposed to varying environments at different points [8]. Using the nondestructive testing method, the thickness of the walls was determined to be sufficient to employ the internal spray-lining technique. The utility considered many spray coatings for the rehabilitation of cast-iron pipe. Cement mortar linings were considered an older technology associated with leached metals and lime under certain conditions and low control over mortar composition. Epoxy lining systems have been used in the United Kingdom, Japan, and other countries for many years with favorable results, including

Table 1 Composition of the Edmonton water distribution system

Material of Pipe	Length of Pipe	
	(km)	*(miles)*
Asbestos cement	1,096	681
Cast iron	840	522
PVC	748	465
Steel	109	68
Concrete	106	66
Other	77	48

increased hydraulic performance. It was therefore decided to undertake spray lining of cast-iron mains in two phases in some locations of Edmonton. The two-component epoxy coating, Nitoline WP® manufactured by FOSROC International Ltd., was selected for this purpose. This product has been used successfully in the United Kingdom, New Zealand, and the city of Chateguay in Canada for the rehabilitation of cast-iron water mains [9].

Although it is known that thicker coatings provide better corrosion protection [6, 10], few studies have focused on the strength enhancement that may be provided by a thicker coating. A recent study on the strength of spot repairs using epoxy coating in concrete wastewater pipes showed wide variations in performance caused by differences in pipe surface, since the study employed service pipes excavated from the ground [11]. Another study, conducted by Chemtron Technology and Engineering, California, USA, investigated the use of an epoxy lining in steel pipe to protect holes of $^1/_{16}$ to 3 in. in diameter. The internal hydrostatic pressure was increased continuously until 2,000 psi was reached or when the lining failed, whichever occurred earlier. The short-term tests indicated that the epoxy coating TR-2000 was able to withstand internal pressure of up to 2,000 psi when the epoxy-covered holes in the steel pipe were less than $^3/_4$ in. in diameter. A similar technique was used in the present study to measure the semistructural strength enhancement provided by the Nitoline WP® epoxy system. Semistructural strength is defined here as the ability to bridge over small cracks and pits and protect underlying material, using the strength of adjacent material.

The objectives of this work were to evaluate the semistructural strength offered by the epoxy lining and to measure the amount of regulated chemicals that may leach into water. The scope of the project was limited to testing for volatile and semivolatile compounds that are regulated in Canada for drinking water. The maximum thickness of the lining tested was restricted to 2 mm (80 mil).

MATERIALS AND METHODS

The project consisted of three parts: pressure tests, full-scale field tests, and laboratory-scale leaching studies.

Experimental Design

The authors decided that results from a longer-term test would be more reliable than those from a short time-to-failure test; therefore, the test pressure was restricted to and maintained at 120 psi for 2 hr. Although there are no standard procedures for this particular type of hydrostatic testing, ANSI/AWWA C600-93, which outlines a standard hydrostatic test for the installation of ductile-iron water mains, was followed whenever possible [12]. Accordingly, the hydrostatic pressure was maintained for 2 hr at 1.5 times the working pressure of 80 psi in the distribution system. Class 150 ductile-iron pipe used for water distribution mains was cut into 0.6-m (2-ft) long sections. The length of the pipe sections was selected to satisfy the criterion for effective length of a cylindrical shell. In order that a load at one location does not affect another location, they must be separated by an effective length where the effects fade out to 0.19% of the original magnitude. The rule-of-thumb equation for this effective length is [13]:

$$L_{eff} = 2.355\sqrt{R_m t}$$

Where

t = the thickness of cylinder in mm

R_m = the midsurface radius (nominal radius + half the cylinder thickness) in mm

This equation has an error tolerance of 5%, which is acceptable in most engineering applications. Using the dimensions of the ductile-iron pipe under study, the effective length was calculated to be about 6 cm.

The effective length between the epoxy-covered regions on the pipe sections was about 15 cm, which was certainly sufficient. The length-to-diameter ratio of each section was exactly 3:1 (24:8 in.), which follows a general rule of thumb to reduce end effects [14]. Ideally the ratio must be about 10:1 to completely eliminate end effects [15]. The ASTM Method D 1598–97 for Time-to-Failure of Plastic Pipe Under Internal Pressure specifies a minimum length of not less than three times the nominal outside diameter for pipes larger than 6 in. in diameter [16].

Earlier studies have indicated that improperly cured pipe coatings are a source of chemicals leaching into water [17, 18]. In 1982, an epoxy coating was found to release small amounts of benzyl alcohol even six months after lining [17]. Epoxy coatings have since overcome this problem. In the present work, full-scale field tests were carried out to screen for chemicals leaching out of the epoxy coating. In these field tests, the cast-iron water mains were internally lined with Nitoline WP® epoxy resin, following the standard procedure outlined in "In Situ Epoxy Resin Lining—Operational Guidelines and Code of Practice," published by the Water Research Centre, UK. The rig for spray lining was provided by an affiliate of the EPCOR group of companies. Following a 16-hr cure period, closed-circuit television inspection was conducted to check the lining, and the epoxy coating was left to cure at ambient temperature. After air-cure of the coating, the mains were disinfected with super-chlorinated water (about 50 mg/L of chlorine). Flushing of the mains was then continued with potable water and samples were tested for bacterial counts. With a favorable bacterial result on the following day, the network was put back into service. Drinking water samples were then obtained from customers' taps connected to freshly lined water mains, and samples were collected for up to a month.

Independent laboratory leaching tests were also performed at the University of Alberta. The Nitoline® epoxy lining was sprayed into pre-cleaned glass bottles and subjected to leaching with water. The effects of three important factors—temperature, thickness of lining, and type of water in contact with the lining (Milli-Q water of 18.2 MΩ quality and tap water)—were studied. The leaching trend was followed by gas chromatographic (GC) analysis over a period of one month. These tests followed earlier tests conducted on a laboratory scale to identify the chemicals migrating from the coating into the drinking water. These early static tests were conducted by maintaining the epoxy lining in contact with high-purity water and testing the water at a predetermined time. Mass spectroscopy was used to identify chemicals in the water by comparing the peaks obtained to chemical library standards, while GC was used to measure the amounts. Results indicated that no harmful chemicals were present in significant amounts. However, small amounts of toluene (about 6 ppb) were found in the contact water, which was highly pure Milli-Q. Although it was not above regulated limits, the source of the toluene was unknown. Toluene was not indicated on the Material Safety Data Sheets of the product. Presence or absence of toluene in the product could not be confirmed, as the composition of the epoxy resin is proprietary information.

Pressure Tests

For the pressure tests, the pipe sections were connected to receive finished water from the Rossdale Water Treatment Plant situated on the North Saskatchewan River. This was the same water that the customers of the utility received. The average pH of this water was 8.2 and the average hardness was about 116 mg/L as $CaCO_3$. The experimental systems consisted of 0.6-m (2-ft) length sections of 200-mm (8-in.) nominal diameter Class 150 ductile-iron pipe. Each section bore three similar-sized holes or circular discontinuities positioned along a straight line on the pipe, as shown in Figure 1, and separated by a distance of approximately 150 mm (6 in.). The diameters of holes drilled were 20 mm (³/₄ in.), 12 mm

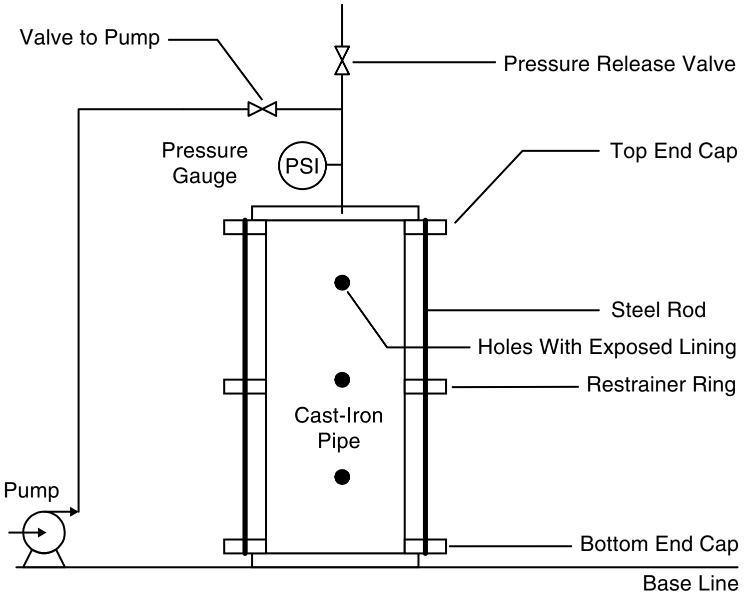

Figure 1 Diagram of experimental setup for pressure tests of pipe sections

(1/2 in.), and 6 mm (1/4 in.). The maximum diameter of the holes was limited to 3/4 in., consistent with the findings of the pressure tests conducted by Chemtron Technology and Engineering, USA. These holes were plugged with putty and a smooth finish was formed on the inner and outer pipe surface. The holes represent corroded pits of similar size that would be covered by the epoxy lining, while the putty signifies corrosion products in the pits. The pipe sections were then internally lined with the epoxy lining under study and allowed to cure. The putty was then removed carefully to expose the lining area beneath. Fourteen lined pipe sections were used in the pressure tests.

Water from the distribution system was filled into the pipe section until overflowing using a hose and air completely expelled from the pipe section using a pressure release valve. Once the system was closed to the atmosphere, a 3,000 psi Jerrycan hand pump with a discharge stroke volume of 11.96 mL was used to pump up the water pressure in the sections to 120 psi. Pressure was increased in small increments (approximately 5 psi every 5 seconds) and monitored using a calibrated pressure gauge with 5 psi sensitivity, 0.5% accuracy, and range of 0–200 psi. The water in the sections was maintained for 2 hr at 120 ± 5 psi using the same hand pump. The pipe sections were tested in duplicate and also triplicate as necessary. The end of the experiment was fixed at 2 hr under pressure or lining failure, whichever came first. Uniform pressure was maintained on the inner wall of the lining, representing the field condition of a water main.

The effects of lining thickness and size of exposed lining were the factors studied using the pressure tests. Lining thickness was regulated by controlling the velocity of the moving spray head and the rate at which the lining material was pumped to it. The National Sanitary Foundation, USA, approves the use of this particular lining at 40-mil (1-mm) thickness in potable water pipes for holiday-free coating [19]. For the tests, the thickness was maintained at 1 mm and sections were sprayed twice to obtain the 2-mm lining thickness as required.

Field Investigations

In the field, superchlorinated water in the mains was tested in anticipation that the first water in contact with the lining would have the highest level of contaminants. Superchlorinated water was flushed out of the mains before the mains were put back into service. During this flushing stage, superchlorinated samples were obtained by allowing the hydrants to discharge for a minute and water was then collected in prepared bottles. An amber bottle of 1-L capacity was used to contain samples to test for turbidity and UV_{254} absorbance. Samples for volatile organic compounds were directly collected into precleaned 40-mL GC vials that contained ascorbic acid as preservative. The superchlorinated water samples were tested for turbidity, UV_{254} absorbance, and volatile organic compounds following methods described in *Standard Methods in the Examination of Water and Wastewater* [20].

After the superchlorinated water was completely flushed out, the mains were flushed again with potable water and put back into service. At this time, drinking water samples were obtained from domestic taps fed from the freshly epoxy-lined mains. These grab samples were taken weekly up to a month after restoration of service. The drinking water samples were tested for turbidity, UV_{254} absorbance, volatile organic compounds, total residual chlorine, and flavor profile analysis or odor test. Field testing was carried out according to the following protocol. The pipes were allowed to cure for a minimum of 48 hr and in many cases for a much longer period. The water mains were flushed with superchlorinated water and flushed again with drinking water. Samples were obtained from five sites as per the sample plan described in Table 2.

The procedure involved opening the cold-water tap in the customer's kitchen and letting the water flow at a moderate rate for 3 minutes before collecting the samples. The kitchen tap was selected in all cases, as it was the most likely source of drinking water for the household. Houses that had water softeners or filters were excluded from the study. As in the case of the superchlorinated samples from hydrants, a precleaned 1-L amber bottle was used to contain samples to test turbidity, total residual chlorine, and UV_{254} absorbance. Another 1-L amber bottle was filled to overflowing with water for odor tests. Samples for chromatographic analysis were collected by filling to the brim of the precleaned 40-mL GC bottles containing preservative. Some superchlorinated samples and those of the first day of resumed service were collected in special 1-L amber-colored bottles provided by the Alberta Research Council (ARC) and sent there for the analysis of semivolatile compounds classified by ARC as extractable priority pollutants. Amber-colored bottles were used to prevent photochemical degradation and thus preserve the integrity of the water sample. All water samples were transported to the EPCOR Water Laboratory in coolers containing ice packs immediately after collection. For comparison, samples were also obtained from a house connected to an unlined part of the distribution system.

All water quality tests performed in these experiments followed the procedures outlined in the *Standard Methods in the Examination of*

Table 2 Field sampling regimen during the test period (number of samples)

	Chlorine, Turbidity, UV_{254} Absorbance	Odor	Volatile Organic Compounds	Semivolatile Organic Compounds
First-day fill (superchlorinated)	2	2	2	1
After flushing, day 1	5	5	5	2
After 1 week	5	5	5	1
After 1 month	5	5	5	1

Water and Wastewater [20]. The instruments used for each of these tests were:

- HACH 2100 N Turbidimeter for turbidity
- Odor panel of five people doing sniff tests for odor
- Wallace and Tiernan Amperometric Titrator (using 0.00564 N phenylarsine oxide as the titrant) for total residual chlorine
- UV 2101 PC spectrophotometer for UV_{254} absorbance
- Varian 3400 GC/FID and Tekmar 3000 purge-and-trap GC for volatiles analysis

Accuracy and precision of the GC was checked using a 200 μg/mL toluene standard obtained from Sigma-Aldrich Canada. Accuracy was determined to be 95% and duplicate runs were within 4% of each other. Other instruments mentioned above are routinely calibrated at the EPCOR Water Laboratory.

Bench-Scale Leaching Tests

Bench-scale tests were used in many early studies to observe leaching of compounds from pipes [17, 21, 22]. Preliminary leach testing of the Nitoline® product was done by the EPCOR Water Laboratory to identify some of the compounds in the leachate. The procedure involved filling an epoxy-lined container with Milli-Q water followed by testing of the Milli-Q water for volatile and semivolatile compounds using a gas chromatograph mass spectrometer (GCMS). This study describes the second and more in-depth research conducted in collaboration with the University of Alberta.

The studies presented were planned to measure any chemicals leaching out from the epoxy by the action of water. To maximize the amount of chemicals leached out, the authors decided against using static experiments as in the preliminary tests. Instead, magnetic stirrers were used in the lined glass bottles to create a shear effect on walls and a mixing action in the contact water. The bottles chosen were of about 15 cm (6 in.) in diameter to allow steady movement of the spray head in and out. Three factors of interest—temperature, type of contact water, and thickness of lining—were studied to follow their effect on the leaching. The factorial design of the experiment permitted the study of factor interactions at the same time. The temperatures specified in Table 3 were chosen to represent the range of water temperatures found in the Edmonton distribution system. Tap water was used to note if the normal conditions and constituents of drinking water affected the leaching. The thickness of lining was varied to detect if increased thickness led to increased leaching of chemicals. Assuming that some chemicals of interest would be identified by means of gas chromatography, the experiment was planned to also study the trend of leaching with time. The experimental design was a 2^3 full factorial with the conditions as follows.

For this experiment, 16 clear, precleaned glass bottles of 4-L capacity fitted with Teflon-lined screw caps were internally spray-coated with epoxy resin manually. Half of these bottles were lined at a 1-mm lining thickness, while the other half were lined at 2 mm. The bottles were then brought to the laboratory, where 4 of the bottles with 1-mm lining and 4 with 2-mm lining were allowed to cure at 4°C. The remaining eight bottles were cured at 20°C. The cure period was set at 24 hr. After the 24-hr cure, 4 of the bottles cured at 4°C and 4 cured at 20°C were filled to overflowing with deionized water purified by a Milli-Q system. Tap water

Table 3 Applicable conditions for chemical testing

Factors	Low	High
Curing temperature	4°C	20°C
Thickness of lining	1 mm	2 mm
Type of water	Milli-Q (18.2 MΩcm)	Tap water

was filled in the remaining 8 bottles. Thus each bottle had a true experimental duplicate subjected to the same conditions as itself. This ensured that each of the eight (2^3) interactions of the three factors was represented in duplicate. Two blanks for tap water and Milli-Q water were maintained as control for the experiment. Zero headspace was maintained in all bottles to prevent loss of volatile compounds from the water.

Water samples were obtained by dipping a 50-mL beaker into each bottle and filling a prepared 40-mL GC vial to the brim. The beaker was rinsed three times with Milli-Q water before and between sampling runs. For each sample drawn, an equivalent amount (50 mL) of tap water or Milli-Q water was reintroduced into the bottle to maintain zero headspace. Appropriate dilution factors were used in calculating the reported results. Samples were drawn once a week for the duration of one month and transported immediately in coolers to the EPCOR Water Laboratory. A sample was also sent to ARC, Canada, to test for semi-volatile compounds. The GC analysis method followed the procedures in *Standard Methods in the Examination of Water and Wastewater* [20].

RESULTS AND DISCUSSION

Pressure Tests

An examination of the lined pipe sections prior to testing showed that in most cases the lining formed a good bond with the pipe and provided a smooth finish on the inside surface. Fourteen of the 32 lined pipe sections were selected for testing, while the rest were rejected due to minor cracks in the lining and some detachment of lining from the pipe. The detachment occurred since the lining adhered to loose rust particles without bonding to the pipe. Therefore the need for proper cleaning of the surface prior to lining cannot be overemphasized. In some specimens it was observed that the exposed circular lining areas were not flat but had slight elevations, caused by the pressure of the spray against the soft putty in the lining process. This was, however, considered a minor issue and these specimens were included in the experiments. During the lining process, the specimens were laid with the holes facing upward so that the slumping of uncured epoxy lining would not increase the thickness at that part. The pipe sections were kept at 120 psi by pumping water continuously. However, it was found that the system stabilized after about an hour and the leakage observed at the end caps in all specimens was reduced. Leakage was not measured, as it did not serve the objective of the test. The results of the hydrostatic pressure test are summarized in Table 4.

From Table 4 it is seen that the lining in some sections collapsed even before the maximum pressure of 120 psi was achieved. Triplicates were used for 1-mm-thick lining exposed in 1/2-in. and 1/4-in. diameter, since one specimen burst at 5 psi and the other at 120 psi. During the experiments it was observed that any imperfections in the 1-mm lining reduced its strength drastically. This was indicated by the break pressure of the lining, which ranged from 5 psi to 120 psi regardless of the size of holes covered. Among the lining rupture locations on the pipe, all except one collapsed at the top location, while one collapsed at the middle. It is assumed that the pressure surge with each stroke of the pump may have affected the top location before it dissipated to lower parts of the system. The lining is considered to be of similar thickness at all three locations

Table 4 Results of pressure testing of pipe sections

Sample No.	Lining Thickness (*mm*)	Diameter of Exposed Lining (*in.*)	Hydrostatic Pressure (*psi*)	Time Under Pressure (*hr*)	Rupture Location	% Success
1	1	0.75	105	0	top	0
2	1	0.75	45	0	top	
3	1	0.5	5	0	middle	33
4	1	0.5	120	0	top	
5	1	0.5	120	2		
6	1	0.25	5	0	top	67
7	1	0.25	120	2		
8	1	0.25	120	2		
9	2	0.75	120	2		100
10	2	0.75	120	2		
11	2	0.5	120	2		100
12	2	0.5	120	2		
13	2	0.25	120	2		100
14	2	0.25	120	2		

since the pipes were lying horizontally while being lined. It is seen in Table 4 that the increased thickness of lining definitely increases its collapse resistance or strength. This effect of thickness on lining strength was confirmed by regression analysis of these data done subsequently (see Table 5). This regression used the input factors of break pressure, thickness of lining, and diameter of exposed area and the output values as 0 for lining rupture and 1 for successful resistance.

Table 5 Results of pressure test data regression and ANOVA

Regression	Statistics
Multiple R	0.854
R square	0.729
Adjusted R square	0.647
Standard error	0.295
Observations	14

ANOVA

	df	SS	MS	F	Significance F
Regression	3	2.34	0.781	8,096	0.0035
Residual	10	0.871	0.087		
Total	13	3.21			

	Coefficients	Standard Error	t Stat	P-value	Lower 95%	Upper 95%
Intercept	–0.188	0.310	–0.606	0.558	–0.88	0.504
Pressure	0.00602	0.00211	2.85	0.0174	0.0013	0.0107
Thickness	0.406	0.181	2.24	0.0489	0.0022	0.809
Diameter	–0.692	0.397	–1.74	0.112	–1.57	0.192

The R square value of 0.729, which is close to 1, and the significance F value being about nine times higher than the F-critical value both indicate a significant regression. Thus it is evident that the input factors significantly influence the rupture resistance of the lining. The lower and upper 95% confidence intervals for pressure and thickness do not include zero, which indicates that within this experimental range, these factors are highly significant when compared with the diameter factor. The pressure and thickness factors both have a positive effect on the outcome of successful resistance, as shown by the positive value of their coefficients. Thickness has a greater effect than pressure since it has a higher coefficient, thus confirming that thickness is the most important factor affecting successful resistance or strength of the lining. In this case the coefficients are not used to build a model since it would have limited predictive power beyond the tested experimental range. It is not known whether the lining would exhibit similar behavior under sustained pressure for a longer period of time. For further testing, it is recommended that the pressure be maintained for a longer period of time as in tests involving structural liners.

It was observed that if the liner did not collapse within the first 10 minutes, it would handle the test pressure throughout the remaining test period. Bakeer et al. state that collapse during short-term tests occurs typically within one hour [15]. An issue associated with the strength of polymer lining is the phenomenon of creep. This visco-elastic property of a polymer material describes its loss of strength with time and is exhibited at most temperatures of product use. Further research is definitely warranted in this area to make service life predictions. The most important observation of this experiment is the highly reliable performance of the lining at 2 mm thickness. The second epoxy-lining layer bonded seamlessly with the first one to produce the double thickness. It appears that the next layer resolves the unevenness of the first 1 mm layer caused by irregularity in the substrate pipe. It can be safely assumed that the lining at this thickness would protect corroded pipes from disintegration in normal conditions. Since the pressure of 120 psi is used to test the installation of water mains, this observation can be related to epoxy-lined pipes carrying water underground. These pressure tests can be regarded as an industrial-scale test to evaluate the application and performance of different lining materials having similar product specifications.

From this set of tests, it is evident that 2-mm (80-mil) thickness of lining is preferable over the standard 1-mm (40-mil) thickness, which is the minimum requirement recommended by the manufacturer, FOSROC International Limited, to provide uniform holiday-free coverage for corrosion protection. At 1-mm thickness, the epoxy would still reduce the red water problems. The National Sanitary Foundation, U.S., directs the maximum field-use dry-film thickness per coat of 1 mm (40 mil) for the use of epoxy as barrier material [19]. The difference between applying a single 2-mm-thick lining and two consecutive 1-mm linings was not examined; therefore it is not known if the results would differ for the single spray mode. The cost of additional material used in lining may be offset by the reliability of lined pipe. The cost of spray lining is approximately CAD $125 per meter, while that of a single spray coat is CAD $25 per meter. Applying the second coat after the 16-hr cure time would therefore increase the cost by CAD $25 per meter, not including other working costs. However, care must be taken to ensure the thickness is not beyond 2-mm nominal thickness. Slumping is a potential problem with applying the epoxy resin too thickly. Early cracking, blistering, and loss of adhesion may also occur with the application of too great a thickness [23].

Field Investigations

The objective of these tests was to determine if the epoxy lining released any trace compounds of concern into drinking water. The semivolatile organic compounds analyses at Alberta Research Council, Canada, looked for the presence of 53 chemicals that are classified as extractable priority pollutants. Gas chromatography was used to identify 23 volatile organic compounds, especially trihalomethanes, whose amounts in drinking water are regulated. Thus, a total of 76 chemicals were screened in the 41 samples obtained during field tests, which include the samples from hydrants. Most of these 76 chemicals were not found in the water samples. Table 6 provides the maximum concentrations of the tested compounds observed above method detection limits (MDL) during the entire testing regimen and compares them with existing and proposed legislation in Canada [24] and the US [25].

Some of the semivolatile compounds in Table 6, such as diethyl phthalate and di-n-butyl phthalate, are not currently regulated in drinking water by Health Canada. Hence, no guidelines were available for comparison of these compounds. Polycyclic aromatic hydrocarbons such as benzo(a)antracene are also not regulated under the guidelines. The MCL values given for the compounds are enforceable standards that must be met by all operating water utilities in the US. The US Environmental Protection Agency has stated that some chemicals in drinking water are currently being monitored under its Unregulated Contaminant Monitoring Regulation [25]. Industry sources, however, indicate that some of these semivolatile compounds are likely to be regulated by health authorities in the near future.

A few superchlorinated water samples obtained from the hydrants indicated traces of semivolatile compounds, as seen in Table 6. This superchlorinated water was used for disinfecting water mains and was flushed out as per operational procedure. The average value of turbidity for the superchlorinated samples was found to be 10.5 ntu, while the average UV_{254} absorbance was 23.9 Abs/m. High turbidity was observed in the hydrant samples, but not in the drinking water samples from customers' taps. The procedure of flushing is known to disturb sediments in the distribution system, both in the hydrants and in the surrounding pipe network. The superchlorinated water is used to disinfect the mains and is not delivered to customers.

Drinking water samples obtained on the first day in service did not contain any chemicals above established aesthetic limits, as indicated in Table 6. The drinking water samples met regulatory requirements, and average values of 0.45 ntu turbidity, 1.76 Abs/m UV_{254} absorbance, odor intensity of 0.58, and 1.6 mg/L total residual chlorine were obtained for the entire field testing, which fall within normal ranges seen in the drinking water. Similar values were obtained from the house connected to an unlined part of the distribution system. The odor descriptions ranged from sweet, musty, and sour to chlorine and solvent-like smells. Consistently low values (on a five-point scale) were given to all odor samples. The sample obtained after a month in service was tested at ARC, which showed no unusual peaks or compounds, ruling out the possibility of leaching of the resin constituents during this period. Toluene was found in the lined bottles but not in the drinking water from the field, and this anomaly is discussed in the following section on bench-scale leaching tests.

By all the standard tests described in the "Materials and Methods" section, the water was of excellent quality. Following the lining program, no red water complaints were received by the utility. Based on the tests detailed above, the epoxy lining is verified as being safe for contact with drinking water. For further tests, a more corrosive environment may be used, since it may better simulate the actual conditions of aging and deterioration of the lining material.

Bench-Scale Leaching Tests

As seen in Table 6, toluene was the only volatile compound identified that was not present originally in the contact water. Thus it was the only one of the 23 volatile compounds tested that leached solely from the lining.

Table 6 Results of chemical tests with epoxy-lined bottles and field tests

	Canadian Drinking Water Guidelines [24]		USEPA Drinking Water Standards [25]	Maximum Concentrations Obtained		
					Field Tests	
Chemical Name	MAC	AO	MCL	Bench Tests	House	Superchlorinated
Semivolatile compounds analyzed at ARC (in ppb)						
Di-n-butyl phthalate	n/a	n/a	n/a	0.3	1.0	1.5
Phenanthrene	n/a	n/a	n/a	0.0	0.2	0.0
Pyrene	n/a	n/a	n/a	0.0	0.0	0.8
2,4,6-trichlorophenol	5	≤ 2	✓	0.0	1.9	0.0
Benzo(a)anthracene	n/a	n/a	n/a	0.0	0.0	0.3
Benzo(b)fluoranthene	n/a	n/a	n/a	0.0	0.0	0.2
Bis(2ethylhexyl)phthalate	n/a	n/a	n/a	0.0	0.1	1.8
Chrysene	n/a	n/a	n/a	0.0	0.0	0.4
Diethyl phthalate	n/a	n/a	n/a	0.0	0.0	0.2
Fluoranthene	n/a	n/a	n/a	0.0	0.2	0.9

MAC	Maximum Acceptable Concentration (Canada)
MCL	Maximum Contaminant Level (US)
< x	Less than Method Detection Limit
*	Belongs to class of trihalomethanes, total MAC = 100 ppb
✓	Monitored under Unregulated Contaminant Monitoring Regulation (US)
AO	Aesthetic Objective (Canada)
n/a	Not available
n/d	Not determined

Table 6 Results of chemical tests with epoxy-lined bottles and field tests *(continued)*

	Canadian Drinking Water Guidelines [24]		USEPA Drinking Water Standards [25]	Maximum Concentrations Obtained		
					Field Tests	
Chemical Name	MAC	AO	MCL	Bench Tests	House	Superchlorinated
Volatile organic compounds analyzed at EPCOR Water Laboratory (in ppb)						
Chloroform	*	n/a	n/a	24.5	23.1	10
Dichloromethane	50	n/a	5	0.7	< 0.5	< 0.5
Bromodichloromethane	*	n/a	n/a	1.2	1.3	0.6
Toluene	n/a	≤ 24	1,000	267.4	0.6	< 0.5
Total xylenes	n/a	≤ 300	10,000	2.6	< 1.0	< 1.0
Water quality parameters tested at EPCOR Water Laboratory						
Total residual chlorine (mg/L)	3	n/a	< 4	n/d	1.97	
Turbidity (NTU)	1	≤ 5	< 1	n/d	0.62	
UV_{254} absorbance (Abs/m)	n/a	n/a	n/a	n/d	2.3	
Odor intensity (no units)	n/a	inoffensive	n/a	n/d	1.0	

MAC	Maximum Acceptable Concentration (Canada)
MCL	Maximum Contaminant Level (US)
< x	Less than Method Detection Limit
*	Belongs to class of trihalomethanes, total MAC = 100 ppb
✓	Monitored under Unregulated Contaminant Monitoring Regulation (US)
AO	Aesthetic Objective (Canada)
n/a	Not available
n/d	Not determined

This was in accordance with the findings of the earlier static leaching tests. Laboratory tests also revealed that Milli-Q water showed no aggressive leaching tendencies, while tap water leached no additional chemicals. The test was conducted in duplicate and the average of the duplicates was used for analysis. Usual quality assurance/quality control guidelines of the EPCOR Water Laboratory were practiced during both sampling and testing steps. The trend of toluene leaching obtained is depicted in Figure 2.

The labels are given in the format of temperature(°C)/thickness(mm)/water type. The range of toluene concentration in the bottles was 14.2 to 59.1 ppb on the first day, which spread slightly to 12.3 to 63.1 on the seventh day. The toluene concentration was found to drop after Day 7, as seen in Figure 2. The leaching did not carry on to the earlier extent even after 21 days. One of the bottles, under the conditions 4c/2/MQ, exhibited an exceptionally high level of toluene, as seen in Figure 2. To identify the reason for this, a time-ordered test was conducted to link toluene concentration with the order in which the bottles were lined. Results of this later test showed that the first bottle lined had a very high toluene concentration and the toluene dropped exponentially in the successive bottles.

The source of this toluene was then traced back to its use in cleaning the spray head of the lining rig, which may have caused contamination of bottles. The cleaning liquid was previously identified only as FOSROC Solvent 102 in the product description. Continuous spraying would purge the toluene from the spray head, causing the exponential decrease of toluene seen in successive bottles. The concentration of toluene was reported as either zero mg/L or less than MDL for all field samples of drinking water (Table 6). This is definitely due to the postcure flushing operation that is standard practice in spray lining. When the lined bottles were rinsed out and refilled to correlate the bench testing data with the field samples, the expected concentration drop to near zero was obtained. Water in the distribution system pipes is continually renewed from other sections, unlike the bench-scale test setup. Toluene is not expected to be a problem in real-life scenarios, because any toluene contamination from the spray head during lining would be flushed out with the superchlorinated water in the initial flushing.

The factorial design of the experiments was based on the assumption that some volatile chemicals would leach from the lining and the effects of the experimental conditions on this leaching would be studied. However, none of the tested chemicals leached from the lining itself. Toluene, which was selected as the indicator compound based on earlier

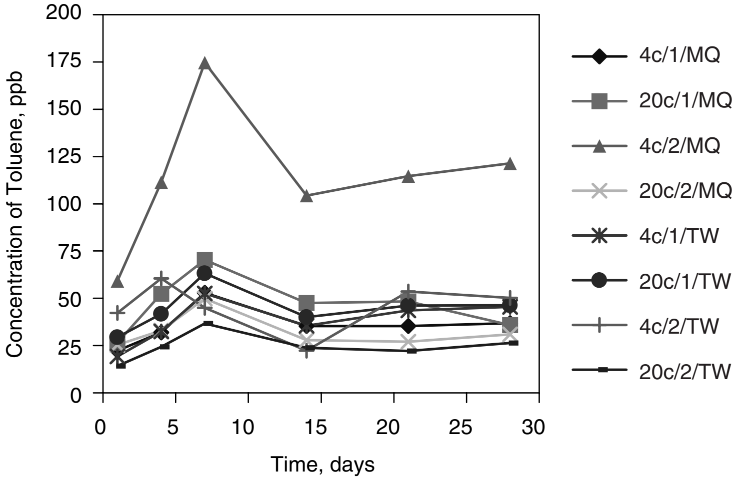

Figure 2 Cumulative results of toluene leaching for all 28 days

static tests, was established to be a contaminant that is not an integral part of the epoxy lining. The toluene concentration in the bottles was not influenced much by the subsequent conditions of cure. The trend of leaching for most of the combinations of conditions followed the same pattern (Figure 2); thus the temperature during cure did not seem to affect leaching. This implies that the product performs well even under low temperatures. Since the double thickness of lining did not affect leaching, the recommendation from the pressure tests is valid. Also, the normal constituents of tap water did not leach any additional chemicals from the lining. Thus, it may be expected that the lining is nonreactive with normal tap water. The leaching of toluene appears to be largely a surface phenomenon, since the concentration of toluene was observed to drop after 7 days. Since no other chemicals of concern were found during the initial GCMS tests, it may be safely assumed that the epoxy lining serves the purpose of internal corrosion protection while complying with drinking water regulations.

CONCLUSIONS

The epoxy lining sprayed at 2-mm (80-mil) thickness provided protection to the underlying corroded pipe, with pits of up to 3/4 in. in diameter, from further deterioration under normal service conditions. This is seen from the ability of the exposed lining in this diameter to withstand an internal water pressure of 120 psi for 2 hr. The lining at the standard 1-mm (40-mil) thickness was shown to protect corroded areas up to ¼ in. in diameter with a 75% efficiency. At this thickness, the lining provided uniform coverage of the pipe walls, based on visual observation during the postcure camera inspection.

Field samples of drinking water met and surpassed the Canadian regulatory requirements. Toluene found in the bench tests was not present in any field water samples from customers' homes, indicating its removal during air-cure and subsequent flushing. Since about 50% of the pipe samples were rejected before the pressure test, thorough cleaning of the pipe prior to spray lining is very important. The epoxy lining tested has proved to be a reasonably good rehabilitation system judging by its physical and chemical characteristics.

RECOMMENDATIONS

Long-term pressure tests may be conducted to study changes over time in semistructural strength characteristics of the epoxy lining. Accelerated testing of the lining under different exposure conditions may be useful in service life predictions.

ACKNOWLEDGMENTS

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CHAPTER · 20

Introducing Membrane-Treated Water Into Distribution System Without Upsetting Existing Pipe Scale: Scottsdale's Approach

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ABSTRACT

The City of Scottsdale, Ariz., manages a Centralized Groundwater Treatment Facility (CGTF). The source water to the CGTF comes from four production wells. At this facility, the City employs air stripping to remove trichloroethylene (TCE). The design capacity of the CGTF is 13.5 million gallons per day (mgd) and the average operational flow is approximately 11.5 mgd. The total dissolved solids (TDS) and hardness concentrations of the CGTF effluent are 830 mg/L and 390 mg/L, respectively.

The City is planning to install a reverse osmosis (RO) treatment plant to treat a portion of the CGTF effluent and blend the RO product water with the CGTF effluent to achieve lower levels of TDS and hardness. In addition to lowering the TDS and hardness, the City expects the RO treatment to reduce the levels of arsenic and nitrate in the blended water to well below their target goals of 6 μg/L and 8 mg/L as N, respectively. The existing pipe in the distribution system that receives the CGTF water is galvanized iron pipe, 25–30 years old.

Visual examination of the pipe has revealed varying degrees of corrosion and scaling. There were locations where the internal tuberculation and scale filled almost 50–75% of the pipe opening. An elemental analysis of the scale revealed iron (~80% by weight), calcium (~10%),

silica (2%), and zinc (~2%) to be the dominant compounds. Corrosion of galvanized iron pipe is the source for iron and zinc, while precipitation from water is the source for calcium and silica. The City wants to avoid upsetting the existing scale or excessively releasing corrosion products when the CGTF and RO blended water is delivered through the old, rusty pipes.

Higher iron release was observed in pipes that were exposed to low-pH (7.5) blended waters. Blended water of TDS 550 mg/L had the least amount of iron release. Calcium deposition to scale or dissolution from scale was minimal at pH 8.0 and a TDS of 550 mg/L. Adding blended phosphates slightly reduced iron release and calcium deposition. No visible color increase was observed in the first flush samples for all the water quality conditions that were tested. A pH of 8.0 and a TDS of 550 mg/L are recommended as the optimal water quality parameters for RO and CGTF blended water, to minimize corrosion and scaling in the distribution system.

This chapter outlines a successful approach that other drinking water purveyors can use to identify water quality conditions that would minimize corrosion and scaling when RO treated water is introduced into the distribution system.

INTRODUCTION

Background

The City of Scottsdale (City), Ariz., is a rapidly growing community in the desert Southwest and, like many such communities, faces the challenges of meeting its water service obligations with limited supply options. The southern part of this community (which is also the oldest portion of Scottsdale) has historically been served potable water that is either groundwater or surface water treated by a neighboring city. A significant portion of the groundwater supply comes from the CGTF. This is a Superfund remediation facility designed to remove trichloroethylene (TCE) from the local aquifer via a packed tower air-stripping process.

A process flow schematic for the CGTF facility is provided in Figure 1. The facility is operated through a partnership between the City of Scottsdale, the EPA, and those parties responsible for the contamination, and is expected to function until 2035–2045. The need for water has led this community, like many others similar to it, to explore the available

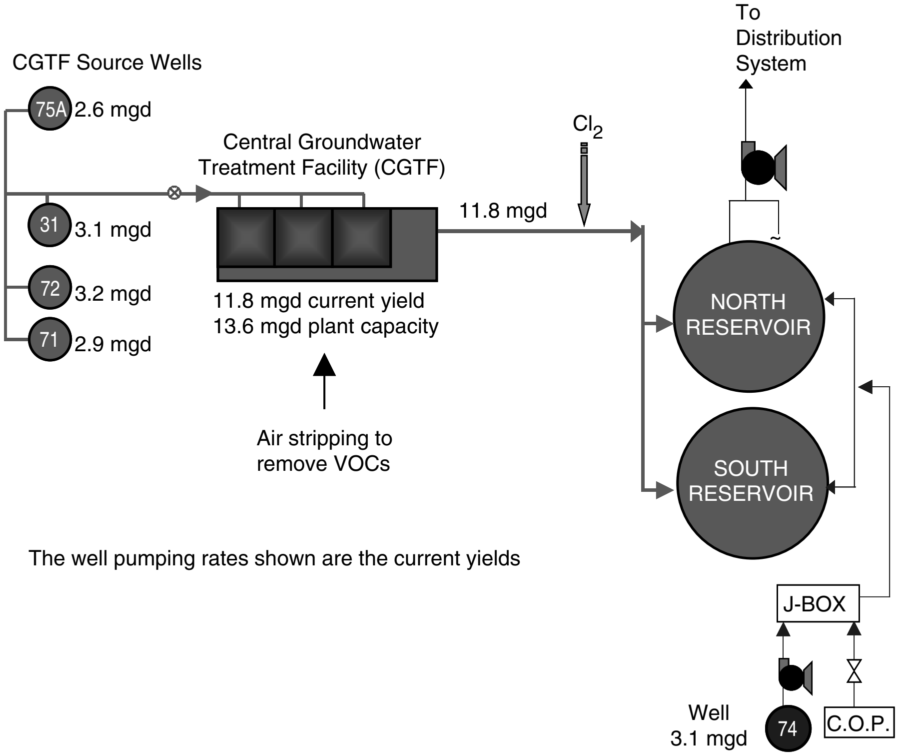

Figure 1 Existing facilities at CGTF

"outside the box" options and to utilize those options that help ensure long-term resource availability for its citizens.

As with many southwestern waters, significant quality issues exist among the wells that supply this facility. Irrigation practices dating back to the late 1800s have contributed significant amounts of nitrate to these waters. Arsenic and fluoride occur naturally, and total dissolved solids (TDS) exceed secondary MCLs for all wells associated with this project. Additionally, these waters contain levels of TCE (resulting from disposal practices in the 1950s) that exceed the current MCL. Table 1 presents the CGTF source and treated water quality data for years 2001–2002.

Problem Statement

The air-stripping process has proven reliable in removing TCE from the contaminated water; however, it also removes carbon dioxide, thus raising the pH and causing calcium carbonate to precipitate in the towers,

Table 1 CGTF source and treated water quality

Parameter	CGTF Source Water*	CGTF Treated Water*
Alkalinity, mg/L	191	180
Arsenic, µg/L	5.6	5.3
Fluoride, mg/L	0.5	0.5
Hardness, mg/L	396	388
Nitrate, mg/L	6	7
pH	7.45	7.14
Silica, mg/L	28.9	28.3
TDS, mg/L	850	855
TCE, µg/L	100–140	Nondetect (<0.5)

*Median concentrations for the data measured over 2001–2002

reservoir, and distribution system. This has resulted in frequent consumer complaints. The City, as part of its master planning process, determined that reverse osmosis (RO) would best meet the primary goal of reducing TDS to an acceptable level and adequately address other water quality issues associated with these waters.

This project was initiated to determine the amount of RO treatment required, evaluate any issues that may impact the performance of the RO membranes, and document the effects this water quality change would have on the existing distribution system. This chapter will focus on the portion of pilot testing that determined the amount of RO treatment required, as well as distribution system impacts that could be expected when the full-scale facility goes on-line.

MATERIALS AND METHODS

Existing Pipe Retrieval, Preservation, and Classification

Approximately 80 ft of existing, $^3/_4$-in. galvanized iron/steel pipe was retrieved from two locations near the CGTF. The pipes were supplying water from the service lines to individual homes and in use for more than 25 years. The direction of flow and orientation of the pipe were marked using spray paint. A visual examination of the retrieved pipe indicated less external corrosion of the pipe under the pavement compared with the pipe under the sidewalk.

The retrieved pipes were preserved by slowly passing CGTF treated water through them. Water was passed in the same direction (upstream to downstream) that was common to these pipes in the distribution system (Figure 2). The retrieved pipes were cut into 8-in.-long segments and color-coded to identify the direction of flow and the location of the segment in the pipe (Figure 3). Based on internal tuberculation and scaling, the 8-in.-long pipe segments were classified into three types:

- Type 1: pipes that were mildly tuberculated and scaled
- Type 2: pipes that were moderately tuberculated and scaled
- Type 3: pipes that were excessively tuberculated and scaled

Figure 4 shows the internal tuberculation/scaling in Type 3 pipe segments. Approximately 30% of the pipe was of Type 1, 60% was of Type 2, and 10% was of Type 3. Type 1, 2, and 3 pipes occurred in the same pipe. At some locations, the pipe had large holes that were covered by an agglomeration of mud and rust. These segments were discarded and not used in the pipe loop study. Type 1 and 2 pipes were found underneath the pavement. Type 3 pipe was found mostly underneath the sidewalk, where the $^3/_4$-in. pipe initially sees the water from the distribution system main. The turbulence in the water may have caused excessive scaling and corrosion. Soil chemistry and structure (porous vs. nonporous) can impact the external corrosion of the pipe.

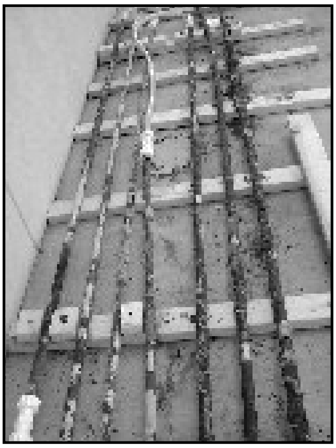

Figure 2 Pipe setup for preserving the retrieved pipes

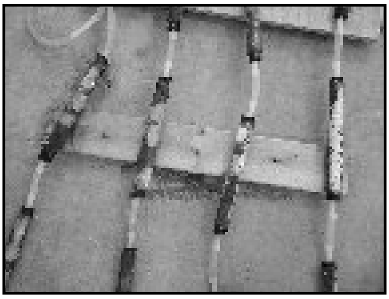

Figure 3 Pipe segments for pilot testing

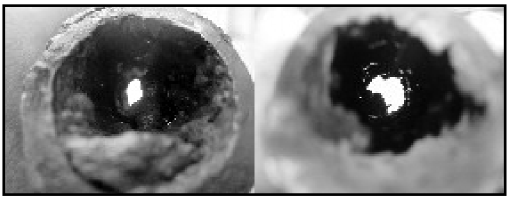

Figure 4 Internal tuberculation and scaling in Type 3 pipe

Baseline Pilot Tests

Eighteen identical pipe loop systems were constructed for corrosion testing. Each test loop had three 8-in. segments that were mounted on a wooden board (Figure 5). All test loops, except test loop 5, had two Type 2 pipes and one Type 1 pipe. Test loop 5 had all Type 3 pipes. Each test loop had an independent feed reservoir that contained the water used for recirculating through the loop at a flow rate of 0.5 gal per minute. To simulate the nighttime stagnation period in the distribution system, water was recirculated for 16 hr and then stopped for 8 hr every day. Samples of water stagnating in the pipes, referred to as first-flush or stagnant samples, were collected and analyzed for metals. To minimize biofouling potential, the feed waters were refreshed on a weekly basis.

For the first two weeks, all test loops were exposed to CGTF treated water at ambient pH (~8.2) and TDS (~832 mg/L) conditions to ensure uniform behavior (in terms of release of corrosion by-products and loss of scale-forming elements) in all the pipe loops.

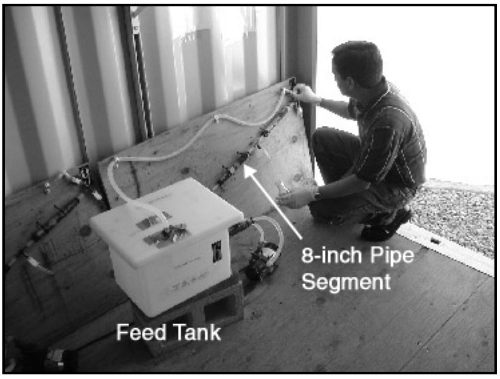

Figure 5 Picture of a pipe loop setup

Water Quality Pilot Tests

Water quality testing was initiated after confirming the uniformity of the loops and that stable conditions existed in each loop from baseline testing. During testing, each pipe loop was exposed to water of unique quality (in terms of TDS, pH, and phosphate). The purpose was to identify the water quality effect on the existing pipes.

A total of 18 pipe loop experiments were conducted over a period of six weeks to evaluate the water quality impact on pipe corrosion and scaling. The pipe loops were exposed to blended waters of CGTF effluent and RO permeate of varying pH (7.5, 8.0, or 8.5), TDS (450 mg/L, 550 mg/L, or 650 mg/L) and phosphate (no phosphate, 30% poly-70% ortho mix, and 30% ortho-70% poly mix). Blending of RO permeate (TDS of ~30 mg/L) with CGTF effluent resulted in the target TDS levels. Feed waters were adjusted on a daily basis to maintain the target pH (by adding acid), chlorine residual (of 1.2 mg/L as free chlorine by adding sodium hypochlorite), and phosphate concentration (of 3 mg/L as total phosphate in loops that had phosphate addition). Table 2 shows the water quality conditions for the matrix of 18 experiments.

Measurements were made to determine the changes (increase or decrease) in concentration of some key elements that are in the corrosion by-products (e.g., iron, zinc) or in the pipe scale (e.g., calcium, silica). Concentrations of corrosion by-products increased (in the stagnant samples compared with the feed samples) as the metals were released from the pipe material. Concentrations of scale-forming elements like calcium, silica, barium, strontium, and magnesium decreased as they precipitated out of the water. In this study, the pipe-water interactions

Table 2 Water quality experimental matrix

18 Water Quality Experiments			
Test #	TDS (mg/L)	pH	Blended Phosphates
1*	~830	8.0	None
2	650	7.5	None
3	650	8.0	None
4†	650	8.0	None
5	650	8.0	None
6	650	7.5	None
7	650	8.0	70% polyphosphate/30% orthophosphate
8	650	8.0	70% orthophosphate/30% polyphosphate
9	550	7.5	None
10	550	8.0	None
11	550	8.5	None
12	450	7.5	None
13	450	8.0	None
14‡	450	8.0	None
15	450	8.5	None
16	450	8.0	70% polyphosphate/30% orthophosphate
17	450	8.0	70% orthophosphate/30% polyphosphate
18§	~50	8.0	None

*Test 1 is a control run with CGTF effluent.

†Test 4 is a duplicate of Test 3.

‡Test 14 is a duplicate of Test 13.

§Test 18 is a control run with RO treated water.

were characterized in terms of "release" of corrosion products from the pipe and "loss" of scale-forming elements from the water.

Copper Corrosion Tests

The City is gradually replacing old, galvanized service lines with copper service lines. Accelerated copper corrosion tests were performed using

3/4-in. diameter, new copper pipe obtained from the City. These tests were designed to form mature scale on the lead/copper surface and allow quantification of each scale's protective nature. Details of accelerated copper corrosion tests were published previously (Edwards and Ferguson, 1993).

In this test, copper surfaces are exposed to the target solution of interest and allowed to corrode naturally for 30 min. Thereafter, the sample was subjected to an anodic polarization (E = +125 mV vs. Ecorr) for 72 hr, forcing the scale to form at an accelerated rate. Anodic current measurements were taken every 15 min for the first four hours and every 30 min thereafter. After 72 hr the applied potential was removed and fresh test solutions were prepared. Each coupon was then exposed to the fresh solutions for 24 hr before corrosion rates were determined electrochemically for aged copper surfaces. The flow rate through the cell was maintained at 0.5 gpm throughout the potentiostatic experiments.

By-product release tests were conducted using the "aged metal samples" after the accelerated testing was completed. Open ends of each coupon were sealed with plastic film so that air was excluded from the coupon interior. The coupons were then rinsed twice with the target solution and filled with fresh solution. Solutions were changed daily over the seven-day test period. Samples of water used to determine by-product releases were collected on the fifth, sixth, and seventh days of exposure after 24 hr of stagnation. "Total metal" by-product release is defined as the total concentration of copper that was released in this sample, whereas "soluble metal" release is the portion of those metals that pass through a 0.45-µm pore size membrane filter.

The tendency of water to cause pitting corrosion is correlated to changes in anodic current density during the potentiostatic test. As the copper is polarized anodically, the actual rate of corrosion is accelerated and scale forms on the copper surface. If the measured corrosion current (anodic current) increases as the test progresses, the surface becomes more active as scale forms and the water is deemed more likely to support pitting attack. If corrosion rates decrease with time, the surface is passivated as scale forms and pitting corrosion is thought to be unlikely. In some cases, no change in current occurs with time and the water is classified as transitional.

The accelerated copper aging experiments were performed using the following blends of CGTF and RO permeate waters:

- CGTF treated water (existing condition control)
- RO permeate water (aggressive control)

- Blend of TDS = 650 mg/L and adjusted pH = 8.0
- Blend of TDS = 450 mg/L and adjusted pH = 8.0
- Blend of TDS = 650 mg/L, adjusted pH = 8.0, and blended poly/ortho phosphates = 3 mg/L as PO_4

RESULTS AND DISCUSSION

Pipe Scale Analysis

Galvanized pipes are coated with zinc to protect from corrosion. Pipes are hot dipped to result in a zinc coating 2–4 mil thick (1 mil = 10^{-3} inches or ~1.2 oz/ft^2). This coating gradually disappears with time, resulting in the onset of corrosion and scaling.

Small quantities of pipe tuberculation and scale were chipped from Type 2 and Type 3 pipes and were analyzed for various elements. The pipe scale was analyzed using (1) acid digestion (USEPA 200.7–9) and (2) Photon Induced X-ray Emission (PIXE). When the scale was being chipped, some iron from the pipe material may also have been chipped out.

Figures 6 and 7 show the results from the pipe scale analysis from Type 2 and Type 3 pipes, respectively. The major constituents in the pipe tuberculation/scale were:

- Iron (~80% by weight; pipe material is the source)
- Calcium (~12% by weight; source is CGTF water)
- Zinc (~3%; pipe protective coating is the source)
- Silica (~3%; source is CGTF water)

Additional constituents that were measured in minor quantities (<1% by weight) were:

- Vanadium (from CGTF water)
- Titanium (from pipe coating)

The PIXE and acid digestion procedures yielded similar results.

Baseline Test Results

In baseline testing, CGTF treated water was recirculated through all the pipe loop systems. All test loops were exposed to CGTF treated water at ambient pH (~8.2) and TDS (~832 mg/L) conditions to ensure uniform

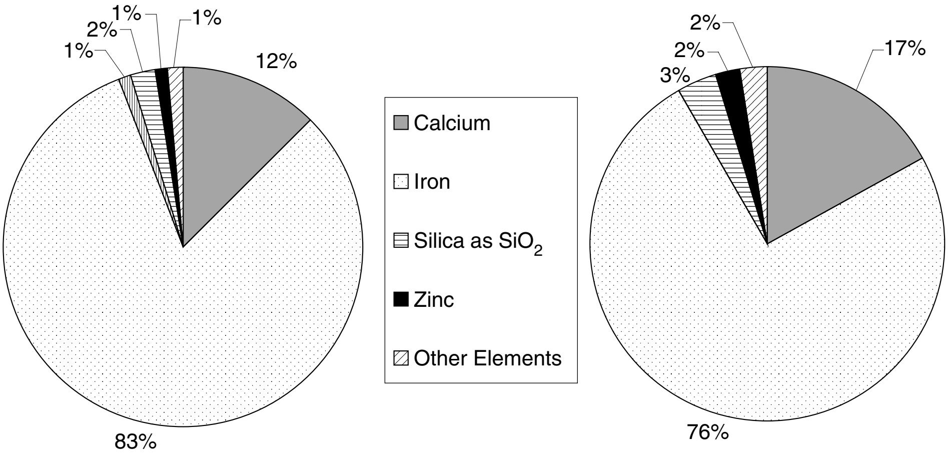

Figure 6 Type 2 pipe scale results

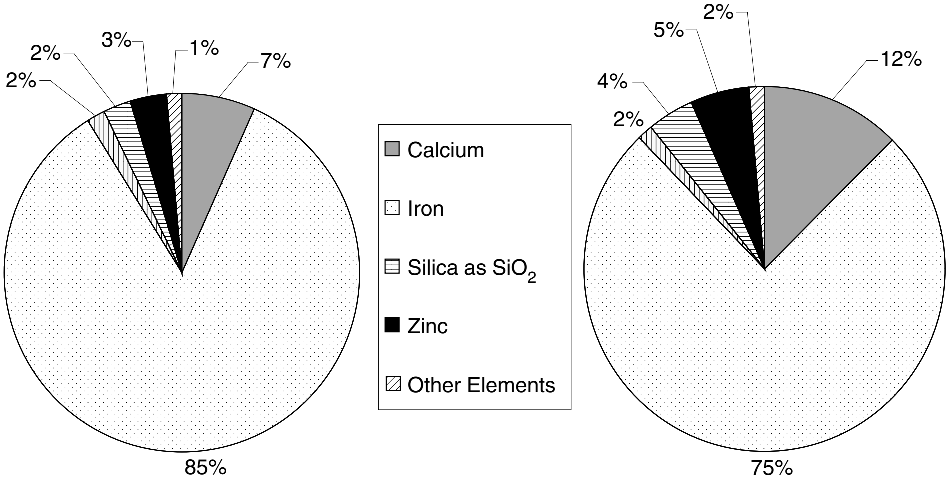

Figure 7 Type 3 pipe scale results

behavior (in terms of release of corrosion by-products and loss of scale-forming elements). Over the course of the two weeks of baseline testing, stable results were observed with respect to iron release and calcium loss for the various pipe types. Type 1 pipes released more iron than Type 2 and Type 3 pipes (Figure 8). The iron-releasing tendency of Type 1 pipes is probably the cause for the fewer tubercles observed in these pipes. More calcium loss was observed in Type 3 pipes than in Type 2 and Type 1 pipes (Figure 9). The existing scale of Type 3 pipe might have served as a nucleation surface for new scale to form. The smoother, Type 1 pipes

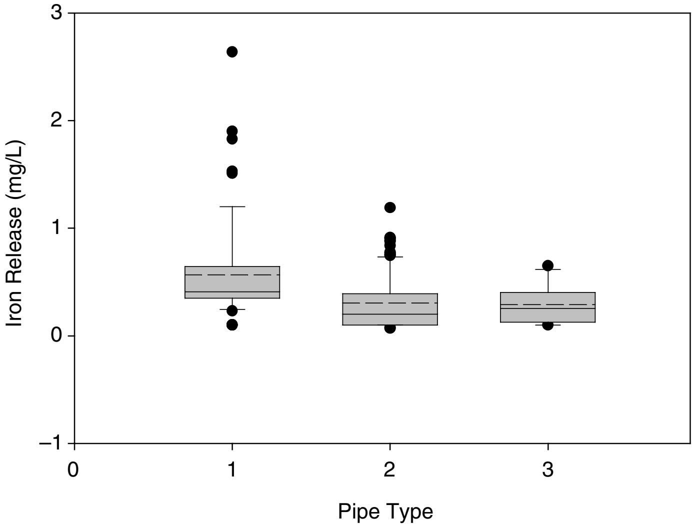

Figure 8 Iron release in various pipe types during baseline testing

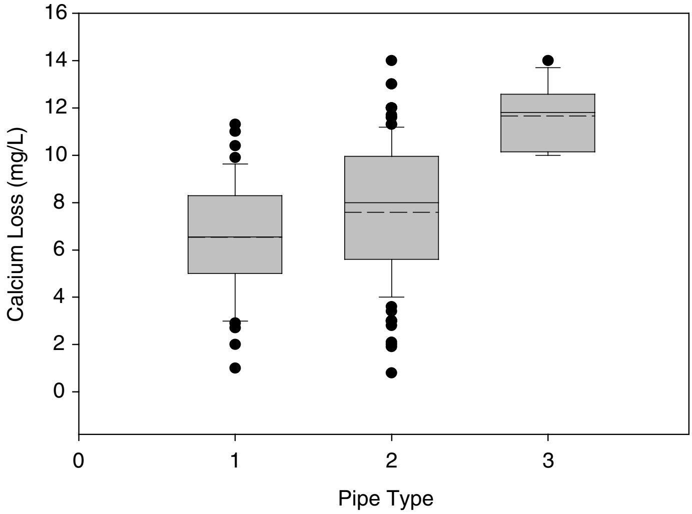

Figure 9 Calcium loss in various pipe types during baseline testing

possibly had fewer locations for scale attachment and growth. Variation of iron releases and calcium losses was minimal at the end of baseline testing. Uniform and stable conditions existed in all the loops.

Water Quality Test Results

Water quality tests were conducted after confirming uniform behavior in all loops based on baseline testing. The impact of blended water TDS, pH, and the use of phosphates on existing pipe corrosion and scaling was analyzed using box-and-whisker plots for iron release, zinc release, calcium loss, and silica loss. Loss of barium, magnesium, and strontium in trace quantities was also observed in the blended waters exposed to the existing pipes.

Lower iron releases were observed in waters with pH > 8.0 and TDS > 550 mg/L (Figure 10). The highest iron releases were observed in waters with low pH (7.5) and low TDS (450 or 550 mg/L). The lowest iron release was observed for blended water with pH of 8.0 and TDS of 550 mg/L. On a mass basis, the zinc releases were lower than iron releases (comparing Figure 11 with Figure 10). Slightly lower zinc releases were observed in waters with pH > 8.0 and TDS < 550 mg/L.

Lower calcium losses were observed in water with low pH (pH < 8.0) and low TDS (TDS < 550 mg/L) (Figure 12). The highest calcium loss was observed in waters with pH of 8.5 and TDS of 650 mg/L. On a mass basis, the loss of silica is comparable to the loss of calcium (comparing Figure 13 with Figure 12). Consistently, lower silica losses were observed in waters with low TDS (< 550 mg/L) and high pH (> 8.0).

Addition of blended phosphates at pH 8 slightly lowered the iron releases (Figure 14). The 30% poly/70% ortho phosphate resulted in lower iron releases than 30% ortho/70% poly phosphate. Similar to iron release results, addition of blended phosphates slightly reduced the release of zinc (Figure 15). Addition of blended phosphates had no impact on calcium loss in the pipes (Figure 16).

No visible color differences were seen in the first flush or stagnant samples collected from pipes exposed to varying water quality conditions (Figure 17). Previous studies related the color of the stagnant samples to the iron concentration and found the threshold limit for noticeable color to be 0.7 mg/L of iron. Iron releases in the stagnant samples in this study were significantly lower than the visual threshold limit (0.7 mg/L) for all test conditions. Therefore no visible color was observed.

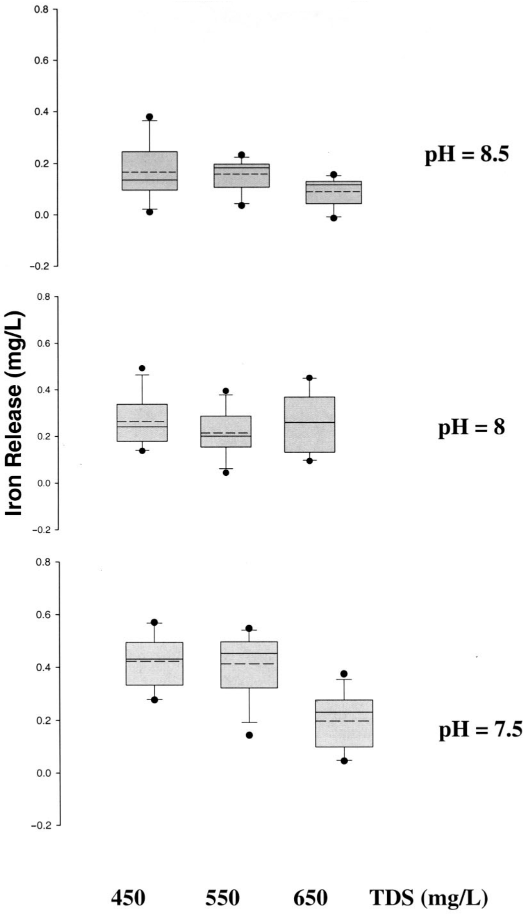

Figure 10 Iron release results for water quality tests

Copper Corrosion Test Results

Thick, mature scales were formed on the surface within a week of accelerated testing (Figure 18). The total corrosion during the test is equivalent to that which would be obtained over two years in typical exposure. Visually, more green deposits were present on the coupons with higher LSIs (> 0.48) (TDS = 650 mg/L and 850 mg/L at pH = 8.0). Very little green scale was present at LSIs of less than 0.20 for waters with TDS = 450 mg/L and pH = 8.0. The addition of phosphate at an LSI of 0.48 also caused more scale deposition.

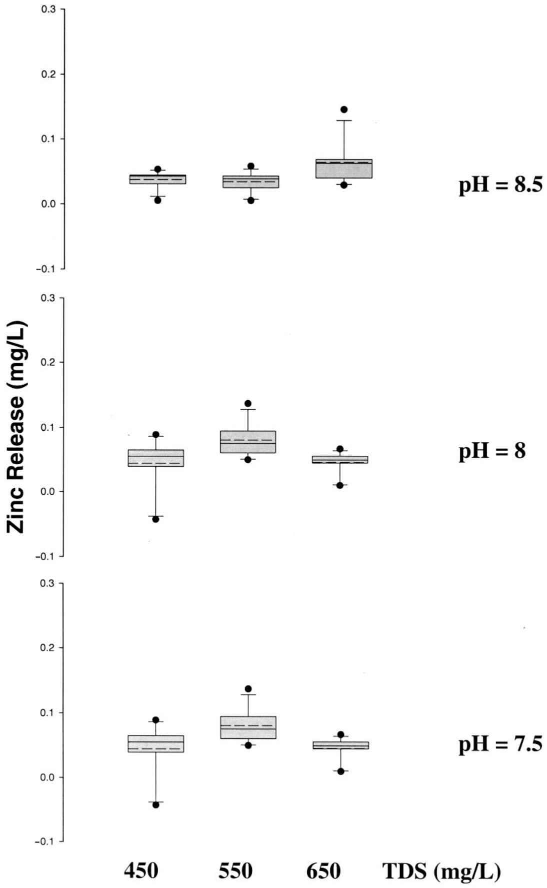

Figure 11 Zinc release results for water quality tests

The corrosion by-products released to each water from the coupon after stagnation were virtually completely soluble (Figure 19). Although the 24-hr stagnation test is relatively stringent compared with that required by the EPA Lead and Copper Rule, and the 95% confidence intervals are wide, significant levels of copper were released to the water in the presence of the aged samples. In particular, levels of copper greater than 5 mg/L were found in the aggressive control (RO permeate) water after stagnation. Phosphate inhibitor reduced leaching by 40% for the water at 0.48 LSI and pH of 8.0 compared with the same water without inhibitor. The best average release was obtained at pH of 8.0 with LSI of

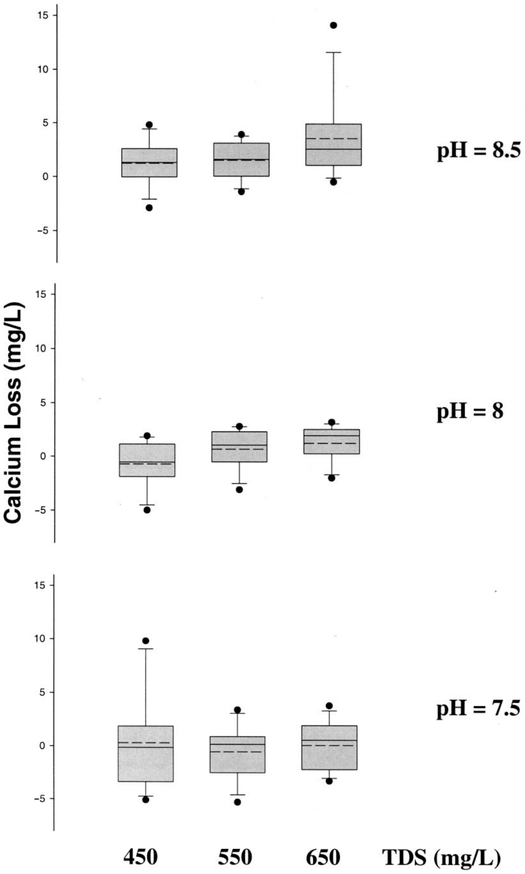

Figure 12 Calcium loss results for water quality tests

0.19, although the existing control is also below the action level of 1.3 mg/L copper.

None of the waters tested caused activation, or higher anodic current rates with time, at the electrochemical potentials (< 150 mV vs. AgCl) deemed to be of interest in this study. The aggressive controls at pH 7.5 were less likely to decrease anodic (pitting) currents with time, and are therefore more susceptible to pitting corrosion. However, pitting does not appear to be a significant concern in this water at pH 7.5 or above given the above data (Figure 20).

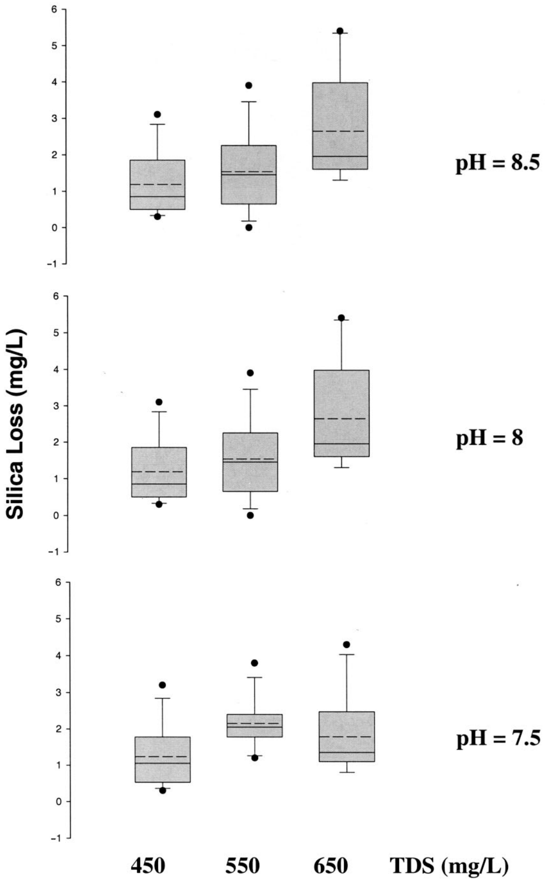

Figure 13 Silica loss results for water quality tests

STUDY FINDINGS AND RECOMMENDATIONS

None of the conditions are deemed likely to cause pitting corrosion of copper. Copper release could be a significant concern if pH is decreased to 7.5 or less, although this could be countered by the addition of phosphate inhibitor. Even at pH of 8.0 in some waters, enough copper was present to suggest a problem with copper release in newer homes. This deserves further monitoring. Since the USEPA's Lead and Copper Rule (LCR) typically selects older homes for monitoring, it is unlikely that the LCR would be exceeded in Scottsdale. However, blue water or metallic-tasting water could be an issue for residents in new homes.

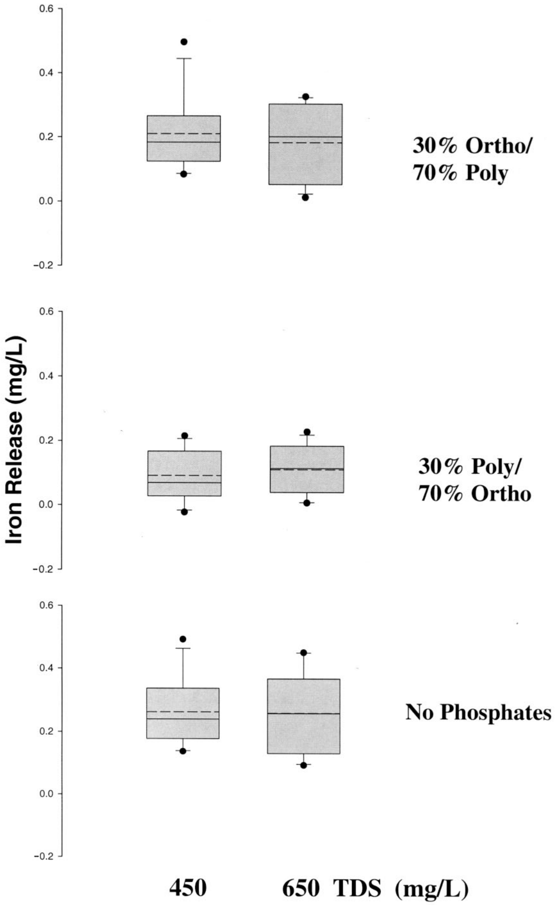

Figure 14 Iron release results for phosphate addition tests

Attention should be paid to customer complaints of metallic-tasting water.

RO permeate of TDS 30 mg/L at an adjusted pH of 8 has the highest potential to dissolve existing scale, as indicated by the negative LSI/CCPP (–2.45/–8.9). The existing pipe in Scottsdale's southern neighborhoods typically contains CGTF treated water with an LSI of 0.64, adjusting the blended water pH to 8 results in LSIs between 0.2 and 0.5 and adjusting the pH to 8.5 results in LSIs between 0.7 and 1.0. It should be noted that LSIs/CCPPs predict only calcium carbonate precipitation or dissolution potential and therefore have limited relevance.

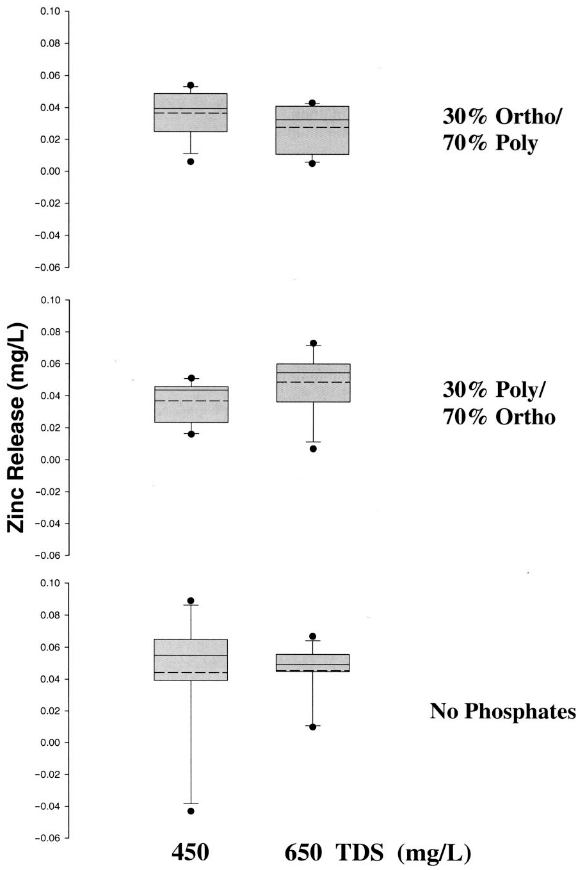

Figure 15 Zinc release results for phosphate addition tests

After reviewing the pipe loop study results, bench-scale test results, and theoretical scale predictors, the following recommendations are made for producing stable water with minimized corrosive and scale-forming characteristics:

- Maintain a CGTF effluent/RO permeate blended water TDS of 550–600 mg/L and a pH of 8.0–8.5.
- Phosphate addition is not necessary and should only be considered in the event of excessive iron release when CGTF effluent and RO permeate blended water is introduced into the distribution system.

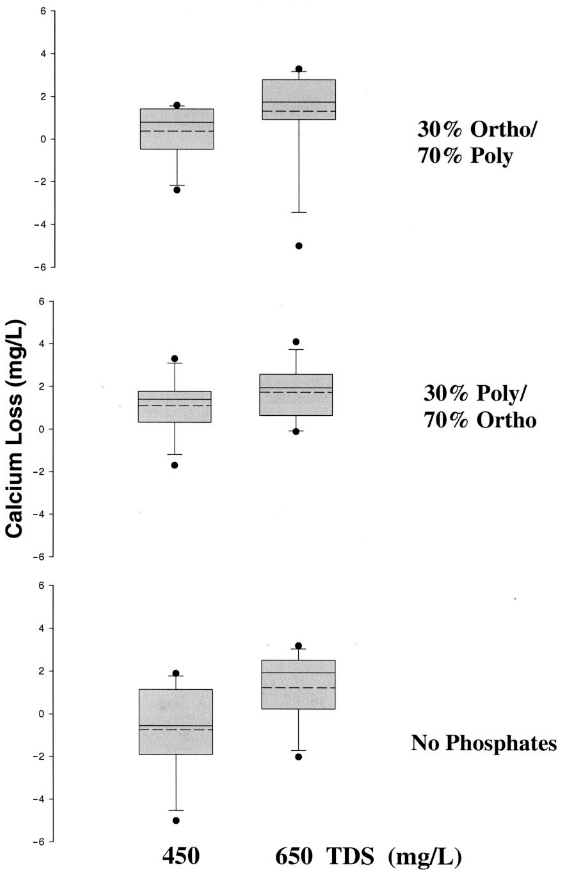

Figure 16 Calcium loss results for phosphate addition tests

To achieve the blended water (CGTF effluent + RO permeate) TDS of 550–600 mg/L, the RO treatment plant should be sized to provide 4 mgd of permeate. The suggested process and blending flow schematic for incorporating RO treatment at CGTF is shown in Figure 21. Blending 6.8 mgd of CGTF treated water with 4 mgd of RO permeate results in:

- Arsenic concentration of 4.2 μg/L
- Nitrate as N concentration 4.5 mg/L
- LSI of 0.34 (at pH 8.0) to 0.86 (at pH 8.5)
- CCPP of 5.2 (at pH 8.0) to 11.2 (at pH 8.5)

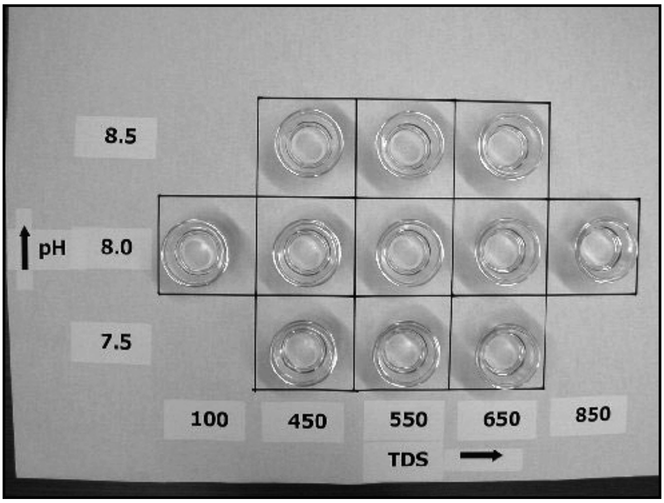

Figure 17 Visual results of blended waters

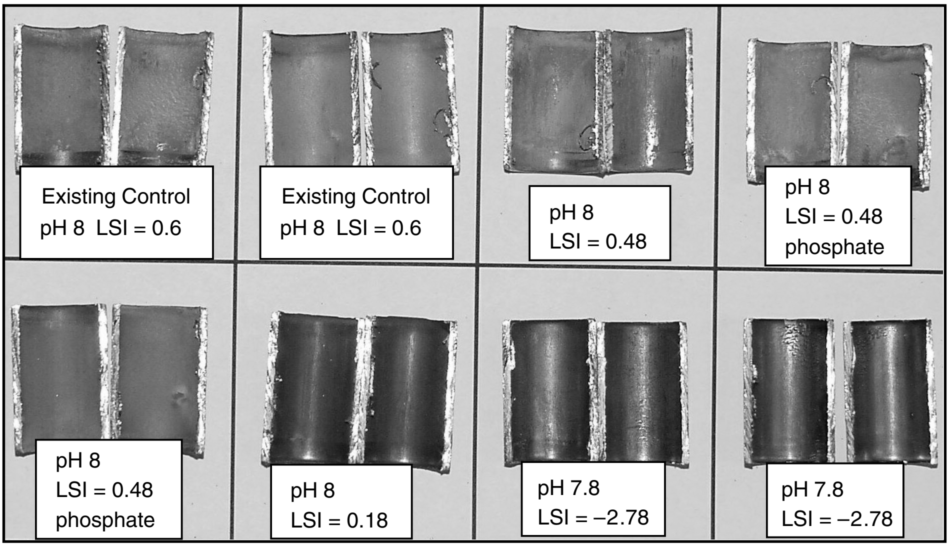

Figure 18 Inside of copper coupons after accelerated testing

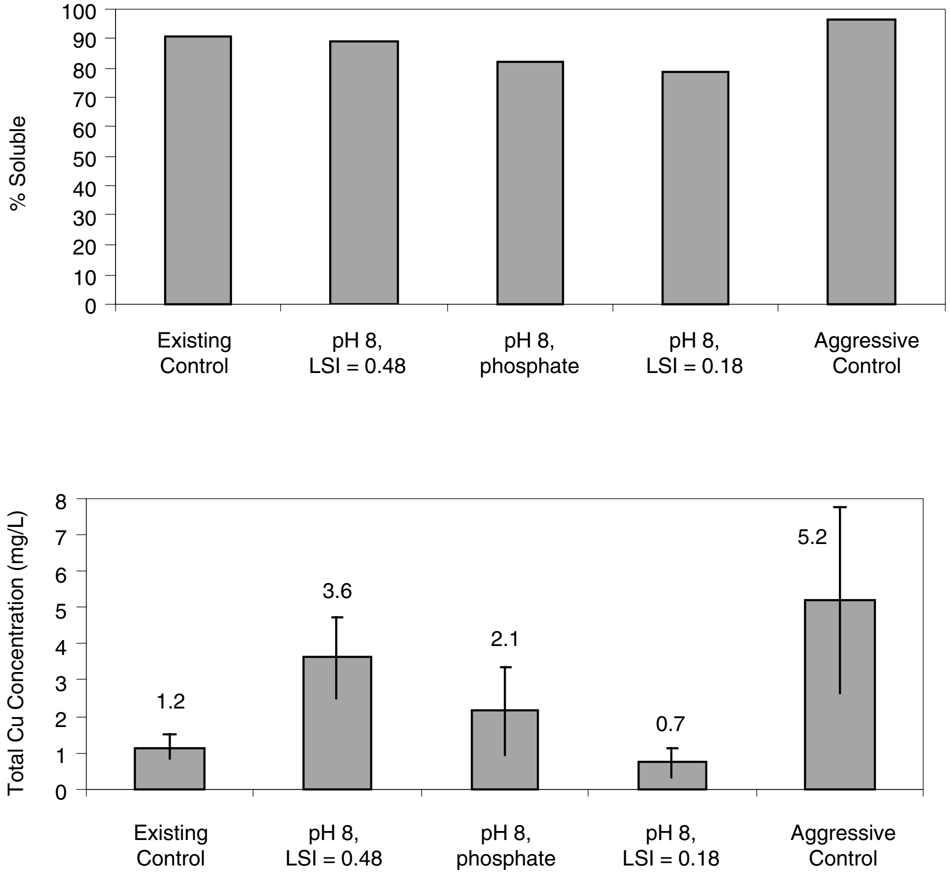

Figure 19 Percentage of soluble copper after stagnation at each condition (above) and total concentration of copper after stagnation (below)

This blended water meets and exceeds all of the City's water quality goals. Details of the RO treatment design are discussed in another paper (Seamans et al., 2003).

ACKNOWLEDGMENTS

The authors would like to acknowledge the following participants in the project:

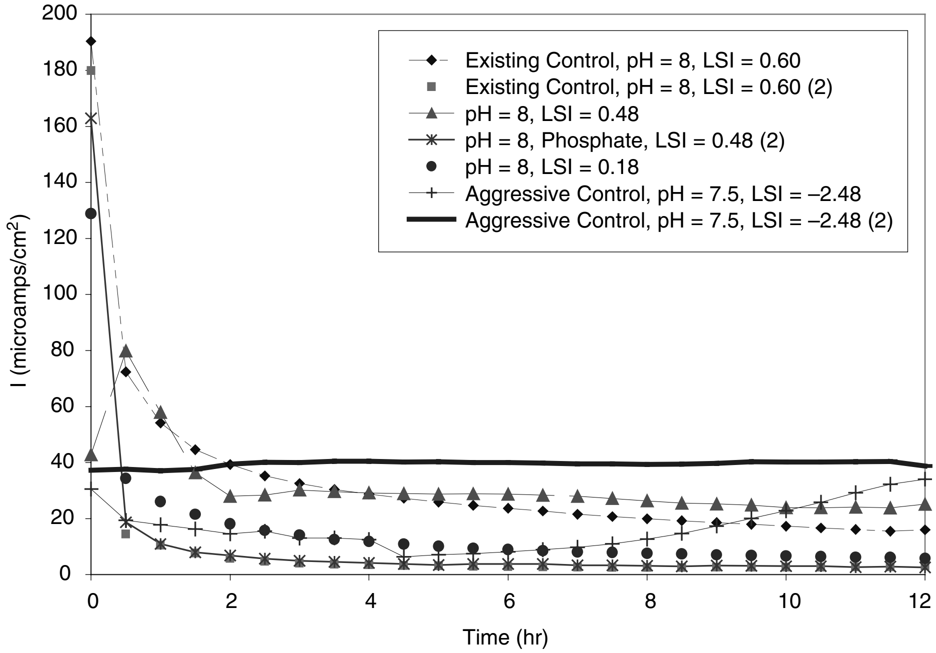

Figure 20 Anodic pitting current versus time trace for Scottsdale waters

- Joe Gross, Dave Petty, and Alison Tymkiw of the City of Scottsdale for their input from time to time
- Suzanne Grendahl, Kimberly Eberenz, and Mark Xervis of the City of Scottsdale for analytical support
- Mark Seamans and Carlos Laborin of CGTF for site coordination and support
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- Jason Rushing, graduate student at Virginia Polytechnic, for copper corrosion test support

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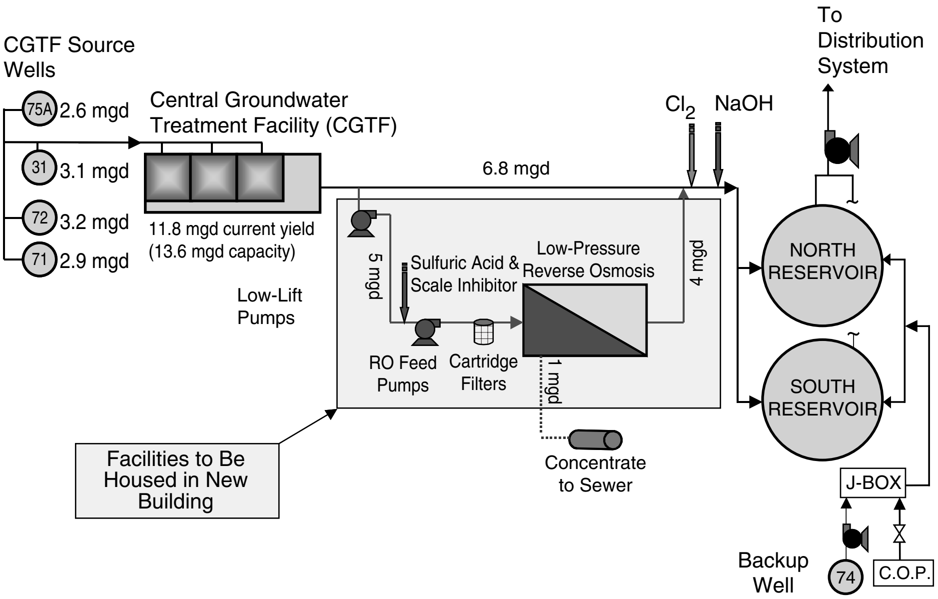

Figure 21 Suggested blending and process flow schematic

2. Seamans, M., Lozier, J. & Kommineni, S. (2002). *Implications and Issues with Reverse Osmosis Treatment of Air Stripped Groundwater: City of Scottsdale Evaluation*. Proceedings of AWWA Membrane Technology Conference, Atlanta, Ga.

CHAPTER · 21

Impact of Chlorine Dioxide on Corrosion in a Model Distribution System

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BACKGROUND

The concern over trihalomethanes in drinking water has led researchers to explore disinfection options other than free chlorine. One disinfectant that is showing promise is chlorine dioxide, a powerful oxidant that has a high disinfection capacity and produces significantly fewer trihalomethanes. However, the impact of chlorine dioxide on corrosion in water distribution systems has not been widely reported. The objective of this study is to examine the impacts of free chlorine, monochloramine, chlorine dioxide, and chlorite on internal corrosion and water quality in a pilot-scale pipe loop system. Results from this study have shown that relative to the free chlorine and monochloramine pipe loops, chlorine dioxide lowers corrosion rates and lowers concentrations of total iron in the effluent. It was therefore concluded that, in addition to being a strong disinfectant, chlorine dioxide could be used to mitigate red water problems in distribution systems.

INTRODUCTION

Interest in the use of chlorine dioxide for disinfection has recently increased because it produces negligible quantities of trihalomethanes (THMs) and offers increased disinfection capacity for *Cryptosporidium* (Chauret et al., 2001). Relatively few studies have examined its ability to suppress biofilm growth and affect corrosion in distribution systems, as originally noted by Aeita and Berg (1986). Recent studies have shown that chlorite (ClO_2^-), which is produced as a by-product during the application of chlorine dioxide (Miltner, 1976), significantly reduced the number of ammonia oxidizing bacteria (AOB) and thus resulted in reduced nitrification in the distribution system (McGuire et al., 1999). Research on possible impacts of chlorine dioxide on internal corrosion and water quality in distribution systems has not been widely reported.

In water distribution systems, internal corrosion can result in degradation of water quality and deterioration of the physical structure of the system (Schock, 1999). All raw and treated waters are corrosive to some degree, and the corrosiveness of water depends on its physical and chemical characteristics (LeChevallier et al., 1993). The impacts from corrosion in water distribution systems can be widespread. It can affect public health, public acceptance of a water supply, and the cost of providing safe water (Schock, 1999). Deterioration of materials resulting from corrosion can result in large expenditures for repair and replacement.

Assessing the impacts of internal corrosion on water distribution systems at full scale can be costly and difficult to perform. Evaluating the physical condition of the distribution system and changes to it is hindered by the inability to examine the system without excavating it (Levin and Schock, 1991). As well, trial runs may result in negative impacts on the water quality. Still, it is important to be able to observe how various strategies impact the distribution system. Researchers have developed several model reactors to experimentally model distribution systems, including annular reactors and pipe loops, both of which have been used to study corrosion in water distribution systems (LeChevallier et al., 1993; Volk et al., 2000). Pipe loop systems are more prevalently used for corrosion studies than annular reactors. Several pipe loops have been developed to simulate water distribution systems, while others have been used to simulate plumbing within a building (Levin and Schock, 1991). Pipe loops can be used alone or in conjunction with coupon tests or coupled with pilot plant investigations. The advantages of using a pipe rig are that the variables affecting corrosion can be systematically controlled and evaluated, and alternate conditions can tested simultaneously under comparable conditions. Pipe loop tests can be used to provide data on many variables, including material selection, pipe life, corrosion rates and types, effects of fluctuations or modifications of water quality, temperature, and alternative treatments.

The objective of the present research investigation is to compare the corrosiveness of three chlorine-based disinfectants on cast-iron pipe through the use of pilot-scale distribution systems. In particular, the three disinfectants to be studied are chlorine dioxide, free chlorine, and chloramines. As well, the impacts of chlorite, a by-product of chlorine dioxide, was examined. It is anticipated that the results of the research will be able to assist water utilities in the overall decision-making process when considering alternative disinfectants. Furthermore, this research will help utilities determine whether longer retention times will have an effect on corrosivity and total iron concentrations in drinking water at the consumer's tap.

MATERIALS AND METHODS

Experimental Setup and Site Description

The pilot-scale testing consisted of five recirculating pipe loops. The five loops were setup so that each of the four disinfectants, plus a control, could be operated in parallel. A typical loop is shown in Figure 1. The test pipe for each loop is a 5-ft length of 4-in. cast-iron pipe sampled from the Halifax Regional Water Distribution System. The approximate age of the pipe is 100 years. A recirculating pipe loop configuration was used as opposed to a once-through flow regime because a previous study using the same pipe and once-through configuration found no observable changes in water quality between the influent and effluent (Maddison et al., 2001). As well, the recirculating system allows retention times of up to 12 hr to be simulated with a pipe velocity representative of a real distribution system. With a once-through system the retention would be under 30 sec with similar velocity. The recirculating pump was sized so that a nominal velocity of 1 fps would pass through the cast-iron section.

The pipe loops were operated at the J.D. Kline Water Supply Plant, which has an average daily flow rate of 100 ML/d. The process train at the J.D. Kline Water Supply Plant consists of pre-oxidation (with $KMnO_4$), CO_2 addition, prechlorination for biofouling control, coagulation (alum), flocculation, dual-media filtration (anthracite/sand), lime addition, and disinfection with free chlorine. Post-filtered water has an average turbidity of 0.03. Post-filtered water that has a pH of approximately 6 and a total chlorine residual of approximately 0.05 mg/L will be used. To reach the desired water characteristics for the project, the water was dechlorinated using a granulated activated carbon column and the pH was adjusted to 8 using sodium hydroxide. As well, polyphosphate was

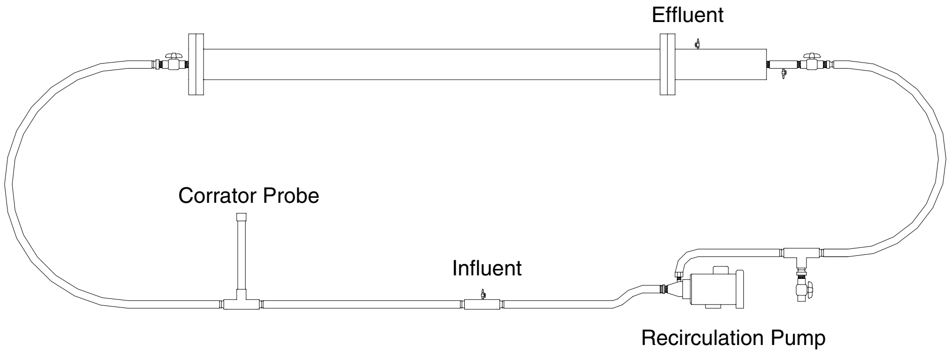

Figure 1 Pipe loop for evaluating corrosion in distribution systems

added to achieve a concentration of 0.8 mg/L for corrosion control, as previous studies have shown polyphosphate offers a desirable corrosion control strategy for the Halifax filtered water (Maddison et al., 2001).

Stock concentrations of disinfectants were fed into the pipe loop using a multicartridge peristaltic pump, which allowed the pipe loops to operate as parallel systems. The stock solutions were changed every 5 days. Monitoring of the effluent residual showed that the decay of the stock concentrations over this time did not severely impact the disinfectant residual in the pipe loops. The chlorine dioxide generator consisted of a bench-top apparatus, in which a 25% sodium chlorite solution was slowly added to a 12 N sulfuric acid solution, producing chlorine dioxide. The chlorine dioxide was purged from the mixture in a gas-washing bottle and trapped in water surrounded by an ice bath. Free chlorine was generated using concentrated solutions of sodium hypochlorite and diluted to the desired stock concentration. Monochloramine was preformed by mixing sodium chloride and sodium hypochlorite in phosphate buffered saline.

Experimental Design

The experimental design matrix is shown in Table 1. The experiment examined two different retention times (i.e., 6 and 12 hr) within the loop and a high and low disinfectant dose for each of the four disinfectants.

The intent of the experimental design shown in Table 1 was to enable utilities to directly compare corrosion rates among disinfectants, as very little information has been reported on the impact of disinfectant residuals on corrosion in iron pipe (McNeill and Edwards, 2001). In

Table 1 Experimental design matrix

Retention Time (*hr*)	Disinfectant Residual Concentration (mg/L)				
	Control: No Disinf.	Chlorine Dioxide	Chlorite	Free Chlorine	Mono-chloramine
6	0.0/0.0	0.25/0.5*	0.1/0.2	0.5/1.0	1.0/2.0
12	0.0/0.0	0.25/0.5	0.1/0.2	0.5/1.0	1.0/2.0

* Low/high target residual concentrations

particular, free chlorine and monochloramine act as benchmarks in the present investigation because there is a general lack of published data for chlorine dioxide in distribution systems.

In addition to new corrosion data, the experimental design examines the possible interaction and main effects of water age (i.e., long retention time) and disinfectant residual on corrosion. For a full-scale system it is very difficult to determine the effect of water age, as the disinfectant residual will decay as the water ages. The effect of water aging in relation to water quality has been demonstrated in several studies (e.g., Prévost et al., 1997) and in particular was demonstrated previously for the Halifax Regional Distribution System (Maddison et al., 2001). In the present investigation, the low and high concentrations shown in Table 1 refer to the target residual concentrations in the effluent of the pipe loop. Thus, the experimental matrix provided in Table 1 was designed to examine the effect of water age explicitly.

Parameters Monitored

Corrosion rates were measured using two-electrode corrater probes. These rely on linear polarization for determining the corrosion rate. As well, the pH, temperature, dissolved oxygen, turbidity, color, and total iron of both the influent and effluent will be recorded. These were measured following methods described in *Standard Methods*. Total iron was measured using the FerroVer method with a HACH DR/3000 spectrophotometer. In the case of dissolved iron, sample water was passed through a 0.45 μm membrane filter. Turbidity was measured using a HACH ratio turbidimeter.

Disinfectant residual was regularly monitored to ensure the target effluent residuals were being met. Free chlorine and monochloramine were measured using the DPD colorimetric method. Chlorine dioxide was measured using the lissamine green spectrophotometric method. Chlorite was measured according to USEPA Method 5510 using a Dionex® 500 ion chromatograph.

RESULTS AND DISCUSSION

Background Corrosion Rates in Control Pipe Loop

Prior to adding disinfectants, the pipes were allowed to condition and stabilize with the base water (i.e., filtered water with polyphosphate addition and pH adjustment). During this period, samples were taken to see if the parameters stabilized. It was noted during this period that there are some discrepancies in corrosion rate, iron levels, and turbidity among the pipe systems, possibly due to variations in deposited material in the pipes. There will be opportunity to compare the effect of disinfectants by normalizing the data for each pipe to the initial conditioned values.

Figure 2 shows a plot of the corrosiveness of the water for the control loop. Each period of the study is indicated on the graph. Similar results have been found for the free chlorine, chlorine dioxide, monochloramine, and chlorite. Corrosion rates remain relatively constant except for a sharp increase in mid-April, which occurred during a blockage to the influent feed line.

Figure 3 shows the total iron levels of the effluent from the control loop. This plot indicates that total iron levels from the control loop remained relatively stable.

Corrosion in Pipe Loops Receiving Disinfectant

In the loops that had a disinfectant applied to them, actually achieving the target effluent residual level was difficult because of the high-oxidant-demand pipe loops and the long response times (e.g., 12 hr) to determine if demands were being met. In general, the target effluent residuals were met (Figure 4). However, there were problems encountered with the monochloramine and for one period, the effluent residual was barely measurable.

Table 2 summarizes the average values for the corrosiveness of the effluent for the free chlorine, chlorine dioxide, monochloramine, and chlorite pipe loops. In general, the corrosion rates increased with the

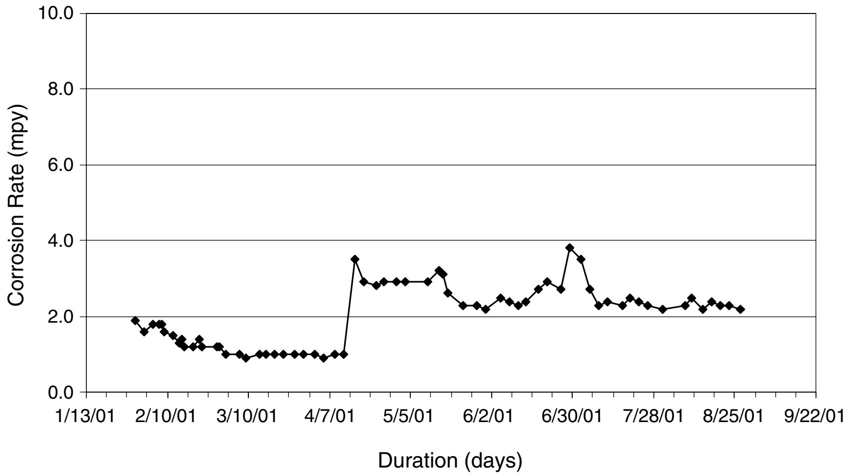

Figure 2 Corrosiveness of effluent from the control loop

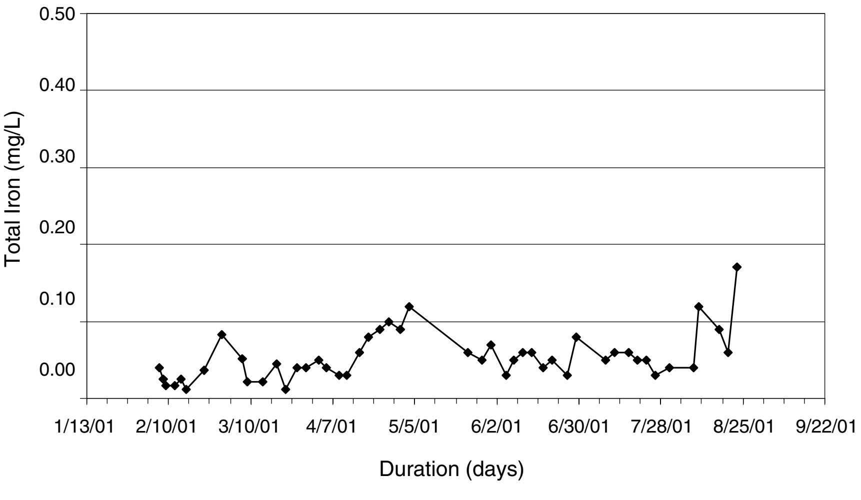

Figure 3 Total iron of effluent from control loop

presence of a disinfectant residual and an increase in the concentration of the disinfectant. This table also indicates that the corrosion rate in the chlorine dioxide loop was lower than in the free chlorine and monochloramine loop. The average corrosion rate measured in the control loop over the entire project was 1.9 mpy.

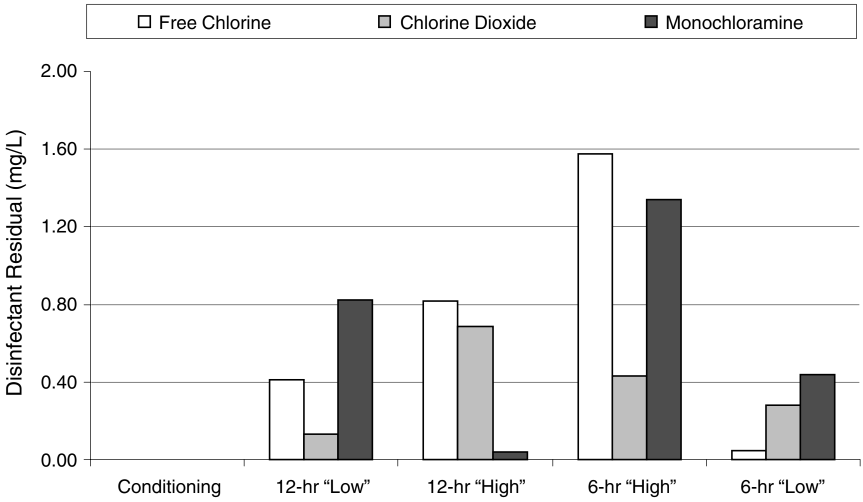

Figure 4 Average disinfectant residuals over the duration of the project

Table 2 Corrosivity of the disinfected water in the pilot-scale distribution system

	Corrosivity of Effluent (mpy)				
		12 Hr		6 Hr	
Disinfectant Treatment	Startup Period	Low	High	High	Low
Free Chlorine	1.3	2.3	4.6	2.9	3.0
Chlorine Dioxide	0.6	1.1	2.3	2.5	2.7
Monochloramine	3.0	4.4	10.7	4.2	4.0
Chlorite	0.2	0.5	1.2	0.8	0.6

The average values for the total iron measured in the effluent from the free chlorine, chlorine dioxide, monochloramine, and chlorite pipe loops are provided in Table 3. In general, the total iron concentrations increased with the presence of a disinfectant residual and an increase in the concentration of the disinfectant. However, this was not always the case. Since the cast-iron pipes being used contained a very thick scale, in many cases the total iron levels were increased when scale broke away.

Table 3 Total iron concentration in the disinfected water in the pilot-scale distribution system

	Total Iron (mg/L)				
		12 Hr		6 Hr	
Disinfectant Treatment	Startup Period	Low	High	High	Low
Free Chlorine	0.22	3.52	5.31	1.27	1.17
Chlorine Dioxide	0.13	0.52	1.00	0.47	0.88
Monochloramine	0.21	0.69	0.38	3.50	0.97
Chlorite	0.03	0.03	0.05	0.02	0.01

The scale was impacted not only by the presence of a disinfectant residual but also by hydraulic conditions within the loop. On several occasions hydraulic conditions within the pipe loop resulted in unexplained incidents of poor water quality. A smaller retention was generally indicative of better water quality. The effluent total iron concentration measured in the control loop over the entire project was 0.05 mg/L.

SUMMARY

Pipe loops are a practical means to assess the impacts of internal corrosion and water quality at the pilot scale without disrupting finished water quality at the full scale. In this particular study, pipe loops were used to evaluate the effects of three different chlorine-based disinfectants on corrosion in water distribution system pipes. In general, it was observed that a longer retention (i.e., 12 hr) resulted in poorer water quality in the free chlorine, monochloramine, and chlorine dioxide loops relative to the 6-hr retention time. The results from the study indicate that chlorine dioxide, known for its high disinfecting capacity, did not significantly increase corrosion in the pilot-scale pipe loop. In comparison with the pipe loops being supplied with free chlorine and monochloramine, both corrosion rate and total iron concentrations were significantly lower using chlorine dioxide. Thus, in addition to being a strong disinfectant, chlorine dioxide could be used to mitigate red water problems in distribution systems.

ACKNOWLEDGMENTS

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CHAPTER · 22

Preventing Red Water Episodes: Integration of New Water Sources

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BACKGROUND

The city of San Antonio is the largest city in the United States that is solely dependent upon groundwater as its drinking water source. Studies have indicated that the current withdrawals from the Edwards Aquifer are depleting this source, which means that the city of San Antonio must find alternative water supplies to reduce existing water use from the aquifer, as well as provide for future growth. The San Antonio Water System (SAWS) is currently pursuing development of new supplies from the Guadalupe River, Carrizo Aquifer, and Trinity Aquifer and has undertaken a Multiple Source Water Integration Study. The purpose of the Multiple Source Water Integration Study is to assist SAWS in anticipating and managing water quality problems associated with introducing water from new sources to the existing water system.

Water quality problems can occur when new water sources are introduced to an existing water system due to changes in baseline water chemistry and changes in system hydraulics and flow patterns. This project will examine one potential problem associated with changing baseline water chemistry: the release of corrosion products and scales that are currently in the distribution system. Corrosion products and scales develop on pipe walls in the distribution system and in household plumbing. Scales can be deposited as minerals precipitate from the water. Corrosion products develop from the piping material in response to the chemistry of the water source. Over time, scales and corrosion products reach equilibrium with the baseline water chemistry and are generally stable. However, when the baseline water chemistry is modified, as with the introduction of water from a new source, the equilibrium is upset and scales and corrosion products respond to meet a new equilibrium and can react in a release of material, causing red water episodes. Red or brown water typically develops from the release of iron corrosion products from older iron piping. It is important to successfully manage re-equilibration to prevent scale or corrosion product release that significantly impairs water quality.

The purpose of the iron release and re-equilibration (IRR) portion of the integration study is to determine the potential for aesthetic degradation resulting from water chemistry changes in older distribution system areas having a higher prevalence of unlined cast- and/or ductile-iron distribution mains, or older homes plumbed with galvanized iron pipe. The IRR uses pipe loop testing with specimens of aged galvanized piping from these areas. Results of the testing program will be used to determine appropriate chemical treatment and/or distribution water chemistry modifications that will ensure the stability of existing iron scales and the minimization of potential red/brown water problems.

This chapter will present the methodology and results of the pilot-scale iron release testing, including the impact of varying hydraulic flow regimes, stagnation conditions, source water blend ratios, and corrosion control strategies imposed on the pipe loops, to assess the impact of different operational and treatment variables. The results of this project will benefit other entities by presenting the practical application of iron-release control strategies.

INTRODUCTION

The San Antonio Water System (SAWS) provides water and wastewater service to approximately 1.1 million people in and around Bexar County, Texas. The SAWS service-area population has grown substantially in the last decade, and current projections indicate that the population will more than double by 2050. Water supply planning studies have determined that even with conservation and reuse mechanisms in place, the SAWS water demands will grow from the current level of about 213,400 acre-ft per year to about 389,500 acre-ft per year in 2050.

In response to the regulatory mandate that the current withdrawal of water from the Edwards Aquifer be reduced, SAWS is pursuing development of initial alternative water supplies from the Guadalupe River, Carrizo Aquifer, and Trinity Aquifer, as shown in Figure 1. SAWS could ultimately utilize several different water sources, including both groundwater and surface water.

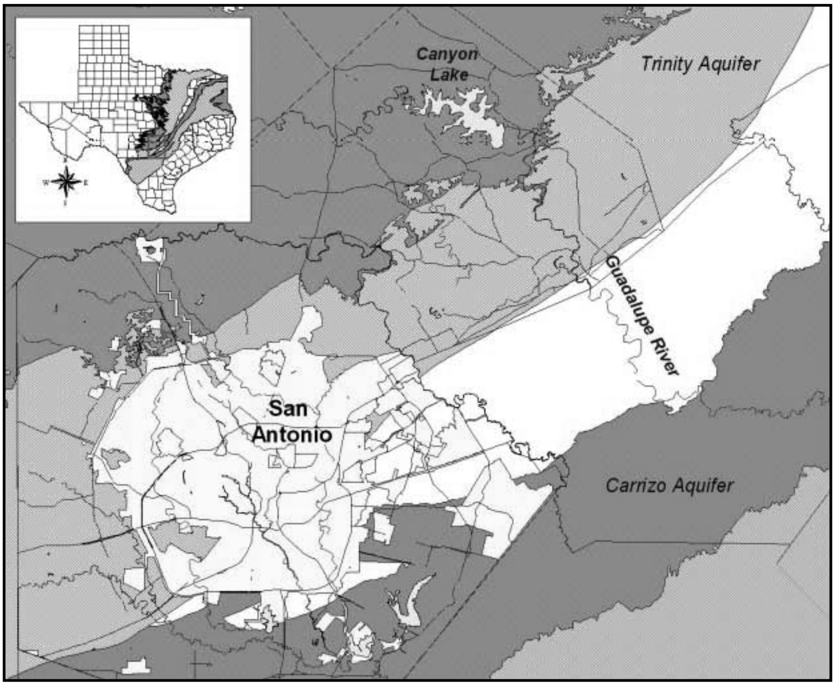

Figure 1 Location of potential alternative water supplies

Water from the Edwards Aquifer is well mineralized with moderate concentrations of alkalinity and hardness as well as other inorganic constituents. Most of the alternative water sources vary significantly in water chemistry, which may include high levels of iron and manganese, low mineral content, and/or moderate concentrations of organic material and bromide. As these alternative sources are introduced or blended into the distribution system, substantial changes in baseline water chemistry within the system will occur.

As a result of introducing new water sources into the system and the baseline water chemistry changing, the pipe surfaces will respond to achieve a new equilibrium. This "re-equilibration" phenomenon may result in the release of iron and other corrosion products that can cause severe aesthetic problems, such as the development of colored water. The most common occurrence is the formation of red or brown water that typically develops from the release of iron from older piping. Blue water can also be formed in some rare instances when minerals are released from the walls of copper piping. Metal release rates of copper- and lead-bearing surfaces can also be higher when exposed to waters with lower mineral content.

The SAWS water distribution system consists of more than 50 pumping stations, 50 storage reservoirs, and 19 million ft of pipe. Many areas of the system contain old, unlined ferrous pipe dating back as far as the late 1800s. All of the piping in the system has undoubtedly developed some accumulations of scale and corrosion products on the internal pipe surfaces. Scales are formed as minerals precipitate from the water in the pipes. Corrosion products develop directly from the pipe material in response to contact with water. These combined surfaces, herein referred to as corrosion scales, have developed over time to a point that they are now in equilibrium with the existing water chemistry. These surfaces protect and passivate the underlying metal surfaces and stop additional corrosion from occurring.

The corrosion surfaces of most concern are those on pipelines made from unlined cast- or ductile-iron pipe and residential plumbing composed of galvanized pipe. In most cases, the galvanization layer will have long since corroded away, leaving a mild steel surface. This type of residential plumbing has generated the most serious re-equilibration problems in other systems, largely because of its proximity to the consumer's tap and the abundance of iron oxide scales that form on the exposed mild steel surfaces.

The SAWS distribution system has developed over the years by constructing many groundwater wells, storage tanks, and pumping stations in response to the area's growth patterns. These distributed supply/pumping centers are largely isolated and tend to serve relatively small areas of the distribution system. Even with the construction of new transmission mains to allow the new sources to hydraulically enter the system, the various source waters will largely be contained within specific areas of the distribution system, but may not be completely zoned off from other sources. As a result, the different source water may blend in the distribution system, resulting in hourly, daily, or even weekly variations in water quality in portions of the system. In addition, as water demands change, the extent to which each source water, or blend of source waters, penetrates the system will change. This oscillation in water quality can be particularly problematic, as it may prevent the effective long-term re-equilibration and passivation of corrosion surfaces. Hence, portions of the system may remain at risk for excessive metal release, colored water, and other aesthetic concerns.

The project methodology was based around a bench-scale testing program that utilizes pipe samples collected from areas within the distribution system that might be impacted by the new sources. These samples were first examined to characterize the composition of the existing corrosion scales. The pipe samples were then utilized in extended pipe loop recirculation studies to quantify the water quality changes that might occur when the new sources are introduced into the system. The loop apparatus was then used to evaluate various water conditioning schemes and develop finished water goals and treatment protocols to mitigate water quality impacts.

APPROACH

It is well understood that water customers will not accept extended aesthetic excursions, such as episodes of objectionable color, taste, or odors. Recurring or chronic water quality problems such as these can turn into public perception nightmares and lead to the loss of trust and consumer confidence. If not managed properly, the introduction of a new water source into a distribution system has the potential to create these types of ongoing water quality problems.

In the SAWS service area, the public has long considered the quality of the Edwards Aquifer to be excellent, and the quality of other water sources has been viewed negatively. In addition, given the basic distribution infrastructure of the SAWS system, it is likely that the new water sources will be contained in generalized areas of the system. Any changes in water quality in definable areas of the system may not be considered desirable by many community leaders.

To address these issues, SAWS developed the methodology for its Multiple Source Integration Study based on the analysis and mitigation of potential for iron release under “worst-case” conditions. It is felt that worst-case conditions will make water quality changes more obvious in the pipe loop studies and provide more challenging conditions for development of mitigation strategies. Also, if water quality changes under the worst-case conditions are addressed, all other areas of the system should remain satisfactory even when field quality conditions vary from those replicated in the bench-scale setting. SAWS recognizes that water quality excursions occur in all water systems, and the study will not eliminate all water quality complaints they might now experience. The goal is to manage the integration process so that widespread, prolonged aesthetic problems do not occur when the new sources are brought on line.

The scope of this study included examination of the three alternative supplies considered to be the closest to implementation. These are

Trinity Aquifer groundwater, Carrizo Aquifer groundwater, and Guadalupe River surface water withdrawn from Canyon Lake.

METHODS

Review Historical Data

The first step in the study was to gather historical water quality data for each of the alternative source waters. These data were gathered from a variety of sources, including the following:

- SAWS
- San Antonio River Authority
- Guadalupe-Blanco River Authority
- United States Geological Survey
- Texas Natural Resource Conservation Commission
- Texas Water Development Board
- Environmental Protection Agency

Raw water samples were collected from each potential source. In addition, treated water was collected from selected water suppliers currently using each source. Analytical testing was completed in the field to measure parameters such as conductivity, pH, temperature, dissolved oxygen, and chlorine (free and total). The water samples were prepared for laboratory analysis, including iron, manganese, calcium, chloride, sulfate, TDS, TOC, bromide, total alkalinity, and color.

These data were summarized along with the historical Edwards Aquifer data in order to characterize each water source. Some of the key water quality parameters are presented in Table 1. These data serve as the baseline water quality for each water source, and will be used to ensure that water samples secured for the pipe loop testing are representative of actual field conditions. Since water quality varies with season and hydrologic conditions, the summarized historical data also provide a context for the broader application of the bench-scale testing results. Note from the data that the Carrizo Aquifer water will require treatment to remove iron and manganese, and the Guadalupe River water will undergo full surface water treatment.

Table 1 Key source water quality parameters

Parameter (*mg/L*)	Edwards Aquifer	Carrizo Aquifer	Trinity Aquifer	Guadalupe River at Canyon Lake*
Alkalinity as $CaCO_3$	279	62	294	179
Total dissolved solids	240	148	321	294
Total hardness as $CaCO_3$	296	121	374	218
Chloride	15	34	76.8	23
Sulfate	8	18	19.3	37
pH	7	6.9	7.3	7.5
Dissolved oxygen	6.2	1.3	2.6	7.1
Iron	–	14.6	0.08	–
Manganese	–	0.19	–	–
Total organic carbon	–	–	–	1.6
Calcium	95	23.3	96	59
Magnesium	14	–	29	17

* Guadalupe River water was treated by conventional surface water treatment.

Collection of Pipe Specimens

The pipe specimens were located by searching the SAWS connection records and examining water meter connection data in the potential impact areas of the system. The records were searched for service connections constructed from galvanized iron pipe that were installed at least 30 years ago. An attempt was made to collect all samples from one location, or in the same neighborhood from houses with similar construction dates. Two nearby residential locations were identified with pipe installed in 1927 and 1930. SAWS field crews contacted each of the customers and replaced the piping at each residence. Approximately 48 linear ft of 1-in.-diameter pipe was collected from the two residences (Figure 2).

To obtain valid results using the pipe loop test, it is very important that the following conditions be achieved during collection of the samples:

Figure 2 Collection of pipe samples from residential service line

- Pipe materials should have similar amounts of interior scaling and tuberculation. This is important so that each sample reacts similarly to water chemistry changes and the differences in iron release can be attributed to water quality differences, not corrosion-scale differences.
- Fresh corrosion of newly cut pipe segment ends must be minimized to prevent newly formed iron release from dominating or influencing the analytical results.

To achieve these two goals, the following preparation/classification processes were implemented:

- The retrieved pipe was cut into 4-ft segments, which is the longest pipe length whose interior can be reliably characterized by observing the interior of each end using a flashlight.
- The pipe segments were first rinsed with tap water to remove fresh iron cuttings, and the cut ends of the pipes were coated

with an epoxy or water-curing urethanes to minimize contact of water and these bare iron surfaces.

- Each segment was then carefully assigned one of three classifications of interior tuberculation:
  - Class 1: Little to nonexistent scaling and tuberculation (almost bare)
  - Class 2: Intermediate level of tuberculation
  - Class 3: Heavy tuberculation (heavy scaling and large tubercles)

Collection and classification of pipe segments continued until adequate segments that were characterized as Class 2 or 3 were secured for constructing the pipe loop apparatus.

The pipe segments were excavated and cut with as little disturbance to the corrosion scale as possible. After cutting, each segment was filled with water from the distribution system and both ends of the pipe were sealed for transportation. The segments were also immediately labeled by address, sample date, classification, and construction date. Flow direction was also noted.

Several smaller pipe coupons (approximately 6 in. in length) were also secured for use in the corrosion-scale analysis. These coupons were prepared as described above, but were not filled with tap water. They were, however, capped to prevent the internal surfaces from drying. For both sets of samples the dry time and disturbance during handling were minimized to preserve the chemical character of the scale.

Analyze Corrosion Scales

The 6-inch pipe sections were shipped to the HDR Water Quality and Corrosion Service Laboratory in Bellevue, Wash., for analysis of the corrosion scales. The purpose of this task was to characterize the chemical constituents and physical properties of the scales as a preliminary step in the assessment of the potential for metal release.

Once in the HDR laboratory, the pipe sections were cut open and the corrosion scales were examined and removed. The corrosion products were then desiccated, weighed, and subjected to analysis. The scale products were also dissolved in strong mineral acid followed by elemental analysis. By nature, this type of analysis is relatively crude and must rely on readily measurable parameters such as hardness, unit mass, and analyses of basic chemistry. Because of the amorphous nature of the corrosion scales, it is not possible to identify the precise mineralogy of the scale constituents (even using X-ray diffraction). Nonetheless, the physical properties of the scale provide useful information on stability and resistance to re-equilibration factors.

Figure 3 is a photograph of the scale on one of the pipe coupons. The physical examination included an assessment of the specific dry mass (mg/cm^2) as well as iron, zinc, and organic content. Assessment was conducted by removing a representative portion of the scale using dental tools, and then sequentially drying, weighing, acid-solubilizing, and analyzing the dissolved constituents.

Chemical analysis of the scale removed from the pipe specimen showed that corrosion rates and the general corrosion environment on the interior of the pipe specimen have generally been mild to moderate. Considering the extended service life (more than 40 years) of the sample, the general interior surface conditions of the pipes are good to very good. While the galvanization layer has long since been lost, the iron-based scale that replaced the galvanization layers are relatively shallow, adherent, and hard. This affords a modest degree of corrosion protection for the underlying steel, resulting in relatively low corrosion and iron release rates. Moreover, the relatively hard scale present on the pipe samples is not just more protective of the underlying metal, it is also more resistant to chemical destabilization, and is capable of handling greater water quality variations without an associated increase in iron solids release.

Figure 3 Corrosion scale from galvanized pipe coupon at 10 × magnification (wet condition)

After acid solubilization, portions of each scale solution were sparged to remove inorganic carbon and then analyzed for organic carbon using a persulfate oxidation and infrared spectroscopic protocol. In most cases the organic carbon content of the scale was below the detection limit of the analytical method. Unlike some voluminous iron-based scales, there was little organic matter or microbial organisms contained in the typical scale from the various service areas. The lack of organic content indicates that the scales are not heavily populated with microorganisms, reducing the possibility of release of organic taste- and odor-causing substances should the scales be destabilized.

The extent of reduction in cross-sectional area by the scale is moderate to low (less than 20%). It is common for galvanized pipe to reduce to levels exceeding 75% once the galvanization layer has been lost. The total reservoir of iron and other mineral mass in the corrosion scale was approximately 24 mg/cm^2. This is a low to moderate amount—a high corrosion-scale mass accumulation would exceed 50 mg/cm^2.

Pipe Loop Construction

Four pipe loops were constructed of 1-inch polyvinyl chloride (PVC) schedule 40 piping. As shown in Figure 4, each pipe loop apparatus consists of the following equipment:

- 13-gal high-density polyethylene (HDPE) sealed reservoir
- 4 gph pump with timer controller
- Flowmeter with totalizer
- Throttling valve
- Check valve
- Three 4-ft pipe segments secured from the distribution system
- Sample ports
- Isolation valve
- Vent valve
- Drain valve

The pump is operated to circulate water through the apparatus at a flow rate of 3.7 gallons per minute (gpm) for 8 hr, and then allowed to stagnate in the piping for 16 hr.

Figure 4 Pipe loop apparatus

Raw Water Preparation

The purpose of the pipe loop testing is to understand the response of pipe scales on pipe surfaces to different finished water qualities; therefore, the waters used should be representative of treated quality as it will be introduced into the distribution system. Since treatment facilities do not exist for some of the alternative water sources, simulation of some treatment was required to approximate finished water characteristics. However, because the testing focused on the water quality reactions resulting from the bulk chemical characteristics of each water, it was desirable to eliminate the impact of oxidation/reduction conditions as variables in promoting water quality problems. Therefore, all waters were aerated to remove any chlorine concentrations and saturated with dissolved oxygen.

For all waters collected, residual chlorine concentration, dissolved oxygen, pH, and temperature were measured in the field at the time of sample collection. Enough water was collected to fill the loops and to stock a supply of makeup water to replace the sample volumes removed for chemical analysis. Additional water was collected as needed during the test period.

Edwards Aquifer water was collected directly from the SAWS distribution system for baseline testing and use for blending with the alternative sources. Water from the Trinity Aquifer was collected during pump tests in the area identified for possible long-term production. The Trinity water was then chlorinated prior to aeration. Guadalupe River water was collected from Canyon Lake at an operating surface water treatment facility. Finished water was secured from the Canyon Lake Water Supply Corporation Surface Water Treatment Facility that operates on Canyon Lake.

Water from the Carrizo Aquifer was collected from existing test wells that were recently constructed by SAWS. A small-horsepower pump was placed into one of the wells and pumped for about four hours prior to sample collection. Since the Carrizo water contains high levels of iron, manganese, carbon dioxide, and hydrogen sulfide, treatment was required to simulate drinking water quality. Since no existing treatment facilities are located in the area, bench-scale treatment was conducted by HDR Engineering. Aeration was used to strip the dissolved gases, accomplish iron oxidation, and achieve dissolved oxygen saturation. A standard aquarium aerator was inserted into the water samples and operated for approximately five days. Residual manganese was oxidized by the addition of about 0.3 milligram per liter (mg/L) of potassium permanganate. The Carrizo water was then pumped through a 5.0 µm cartridge filter to remove the oxidized iron and manganic particles.

Samples taken from each water source before and after aeration (or treatment) were secured and submitted to a commercial laboratory for measurement of the following parameters (as appropriate):

- Sodium
- Potassium
- Total hardness
- Iron
- Manganese
- Chloride
- Nitrate
- Alkalinity
- Turbidity
- Color
- Conductivity
- Silica
- pH
- Dissolved oxygen

To the extent possible, the same parameters were analyzed using HDR's analytical equipment at the pipe loop laboratory to establish the accuracy of on-site testing methods.

Pipe Loop Operation

The pipe loop testing is designed to evaluate the extent of iron release from the existing pipe samples collected from the SAWS distribution system in response to water quality changes. The release of iron from the corrosion scales could be affected by various factors, including the inorganic (concentration of various ions and total dissolved solids), organic (natural organic matter represented by TOC), and electrostatic (surface charge on scales as well as dissolved constituents) quality of water. In order to evaluate the impact of these factors, water quality within the loops will be rigorously monitored and controlled. By imposing different source blends on the loops and tracking iron release, color, turbidity, and other water quality parameters, the chemical parameters of most importance to the stability of existing corrosion scales will be evaluated.

The pipe loop testing was conducted in three phases.

Phase 1 was an initial period of approximately four weeks during which all pipe sections were contacted with Edwards Aquifer water. Stagnant water iron concentrations (and other water quality parameters) were measured to establish the baseline iron-release characteristics of all pipe sections before introducing alternate water sources. Phase 1 was essentially a control period during which pipes exhibiting dramatically high or low iron release could be identified and replaced, and baseline release rates could be determined.

Phase 2 was a period of about two months during which the pipe sections were contacted with different source waters. The combinations of source waters in the four loop apparatuses were as follows:

- Loop 1—100% Guadalupe River (Canyon Lake) water
- Loop 2—100% Carrizo Aquifer water
- Loop 3—100% Trinity Aquifer water
- Loop 4—50% Carrizo Aquifer water and 50% Edwards Aquifer water

This period comprised the main comparative iron-release testing phase of the program. At the conclusion of the first month of testing, the results to date were reviewed to determine if any changes should be made to water quality or the sampling protocol during the final month of testing.

Phase 3 included water conditioning and mitigation studies to determine the most effective and practical treatment to reduce potential iron release in the pilot pipe loops and SAWS system.

The pipe loop operational modes were intended to simulate a home plumbing system that experiences alternating periods of use, then stagnation. The most significant red water problems can develop during stagnation, and then can be observed during the first period of circulation. This is conceptually equivalent to a homeowner who leaves for a weekend trip, returns, and turns on the water tap to observe the first slug of water containing red iron particulates.

The pipe loop test simulated alternating circulation and stagnation conditions. The pump circulating water through the pipe apparatus was outfitted with an electronic timer, which was used to operate the pump for a period of 8 hr (circulation period) and then shut the pump down for 16 hr (stagnation period).

Stagnant samples were obtained at the end of the 16-hour stagnation period. Each vertical section of sample pipe was isolated, and about 50 mL of stagnant water was drawn from the bottom of each section into a clean container. This step is necessary to remove any large debris that may have sloughed off the pipe interior. Upon completion of these analyses, approximately 300 mL of stagnant water was collected from the bottom of each pipe section and composited in a single sample container for chemical analysis. Flow was then reinitiated and air was purged from the system through the vent valves. Chemical analytes tested were as follows:

- Alkalinity
- Total hardness
- Chloride
- Manganese
- pH
- Sodium
- True color
- Apparent color
- Dissolved iron
- Silica
- Dissolved oxygen
- Turbidity
- Conductivity
- Nitrate
- Temperature

Circulating water samples were collected the evening before each stagnation period. These samples were drawn in the same manner as the stagnant samples, and chemical analysis was identical. The total amount of water removed from each loop during each sampling period was about 1–2 L. Makeup water was added to each 13-gal reservoir. It is important to note that each reservoir remained sealed during the testing and was kept as full as possible to minimize dissolved oxygen transfer that might occur in the tank's headspace. Since corrosion products were released into the circulating water, the water in each loop was replaced approximately once per week to maintain the original water chemistry and simulate actual field conditions.

Determination of Iron Release

Iron release rates during the stagnant period were calculated from the difference between iron concentration in the circulating sample and concentration in the stagnant sample that was taken 16 hr later. These data were normalized over the contact area with the pipe specimens to calculate iron release rates as milligrams of iron released per square meter per day (mg/m^2-day).

A mass balance approach was taken to calculate iron release rates assuming the stagnant pipe as a control volume under no flow conditions. The iron release equation is shown below (Equation 1). As written, negative release rates imply that the corrosion scale adsorbed iron. More practically, negative release rates indicate that iron most likely precipitated out of solution and settled to the bottom of the reservoir container or adhered to the walls of the PVC pipe and was not available for analysis in the sample volume.

$$R_{Fe} = (C_{finalsp} - C_{initialsp}) \cdot V_{sp}/(t \cdot S_{sp}) \tag{1}$$

Where:

R_{Fe} = release rate, mg/m^2-day
$C_{finalsp}$ = final stagnant pipe iron concentration (mg/L)
$C_{initialsp}$ = initial stagnant pipe iron concentration (mg/L)
V_{sp} = stagnant pipe volume = $\pi D_{sp}^2 L_{sp}/4000$ (L)
t = stagnation period (days)
S_{sp} = inside surface area = $\pi D_{sp} L_{sp}$ (m^2)

RESULTS AND DISCUSSION

The iron release rates during the stagnation period in all four pipe loops for both Phases 1 and 2 are shown in Figure 5. This figure also indicates the additional test runs that were made after the initial source waters were tested in the loops. This was done in order to assess any apparent impact of the varying pipe loop behavior on the iron release potential of each source water. The additional evaluations included:

- Loop 1—Trinity Aquifer water
- Loop 3—Guadalupe River (Canyon Lake) water

- Loop 4—100% Carrizo Aquifer water

The Trinity Aquifer water introduced into Loops 1 and 3 indicated little change over baseline conditions. The Guadalupe River (Canyon Lake) water introduced into Loop 1 showed an increase in iron release over baseline conditions. Although there was little change from baseline conditions when this water was introduced into Loop 3, the increase in Loop 1 indicates that conditions can exist for moderate iron release.

The iron release rate from 100% Carrizo Aquifer water in Loop 2 indicated that conditions are such that a significant iron release can be expected. The data from Loop 4 for the 50% Carrizo Aquifer blend do not indicate any deviation from baseline conditions; however, when 100% Carrizo Aquifer water was introduced into Loop 4, the magnitude of the release demonstrated was comparable to that seen in Loop 2.

Water Conditioning Studies

After data from the Phase 1 and 2 pipe loop testing were plotted and examined, various water conditioning schemes were evaluated based upon the extent of the iron release caused by contacting the pipes with

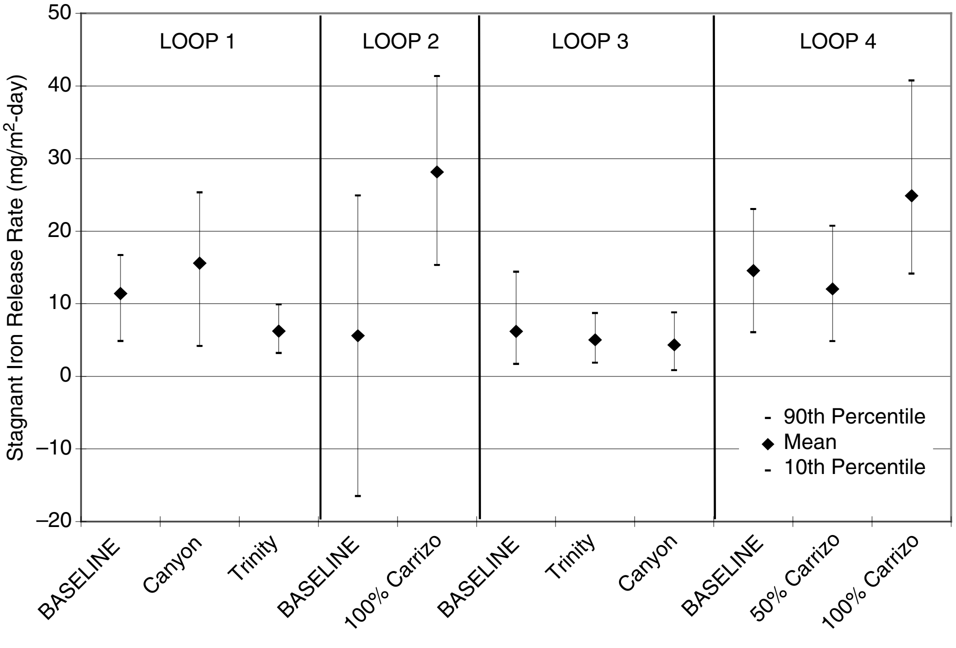

Figure 5 Phase 1 and 2 stagnant iron release rates for all loops

the new source waters. Since the data indicated that Trinity Aquifer water would likely have little impact on water quality, analysis of Trinity water was not continued in the water conditioning (mitigation) studies. After close examination of the Phase 1 and 2 data, the following operational changes were made and several water conditioning schemes were introduced into the loops (Table 2). For each treatment trial, the observed iron release ranges are shown relative to pipe loop and source water.

The results indicate that the addition of polyphosphate effectively mitigated the increased iron release rates caused by the Guadalupe River (Canyon Lake) water. Changes in pH did not appear to have a significant impact on the behavior of the Canyon Lake water. The iron release rates for the Carrizo Aquifer water continued to be relatively high regardless of the mitigation treatment. However, the most promising treatment appears to be the addition of lime and carbon dioxide in an attempt to increase the alkalinity to mimic that of the Edwards water.

SUMMARY AND CONCLUSIONS

The goal of this study was to develop a strategy to manage the integration process so that widespread, prolonged aesthetic problems do not occur when new source waters are brought into the SAWS distribution system. SAWS desired to base the analysis and mitigation of potential iron release under "worst-case" conditions, resulting in obvious water quality changes in the pipe loops and providing more challenging conditions for the development of mitigation strategies.

The results indicate that the Carrizo Aquifer water and Guadalupe River (Canyon Lake) water have the potential to create water quality changes when they are introduced into the SAWS distribution system. The early results indicate that addition of polyphosphate effectively reduces the iron release rates from the Canyon Lake Water. For the impacts of the Carrizo Aquifer water, adjustment of water chemistry (pH and alkalinity) appears to be a promising mitigation strategy. Testing of additional control strategies is now ongoing until the most effective solution can be determined.

SAWS intends to continue these types of integration studies as new water sources near implementation. Future work may also include evaluation of lead and copper release rates, potential for taste and odor changes from the new sources, disinfection compatibility, and public perception management.

Table 2 Phase 3 mitigation and resultant iron release ranges

Loop	Source Water	Treatment	Target pH	Iron Release* (mg/m^2-day)
1	Canyon Lake	Polyphosphate	–	0.9–9.3
2	Carrizo	Sodium hydroxide	9.0	35.6–52.8
	Carrizo	Soda ash	9.0	33.3–51.5
	Carrizo	Lime + CO_2	7.5	15.2–63.4
3	Canyon	Sulfuric acid	7.0	2.7–8.3
	Canyon	Sodium hydroxide	9.0	1.3–4.0
	Carrizo	Lime + CO_2	7.5	4.4–22.5
4	Carrizo	Polyphosphate	–	39.6–73.8
	Carrizo	Orthophosphate	–	42.2–87.9

*Range (10th–90th percentile) of iron release during stagnant conditions

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CHAPTER · 23

Effect of Chlorine on Corrosion

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SUMMARY

The federal Ground Water Rule may require some water utilities that do not use disinfection to begin doing so. A common method of disinfection is to add chlorine to the water. A study was performed to investigate the corrosive effects of chlorine in drinking water systems out of concern for staying in compliance with the corrosion control stipulations of the Lead and Copper Rule. Comparative corrosivity experiments using pipe loops were performed at two test sites. In this study, iron appears to be the most affected by free chlorine addition, followed by copper, followed by lead, which may or may not experience increased corrosion. Elevating the pH of the water at one site (originally at a pH of 7.8, 140 mg/L as calcium carbonate [$CaCO_3$] total alkalinity, 6 mg/L dissolved oxygen) was beneficial in counteracting the corrosive effect of chlorinated water on iron. However, in this case, the elevation of pH was not beneficial in controlling corrosion of lead or copper. Adding orthophosphate decreased corrosion of lead and iron in contact with chlorinated water at the other site (original pH of 7.4, 290 mg/L as $CaCO_3$ total alkalinity, 11 mg/L dissolved oxygen). However, copper corrosion appeared to reach an increased level in the long term.

INTRODUCTION

The US Environmental Protection Agency's (USEPA's) proposed Ground Water Rule (GWR) (USEPA, 2000), with the final version to be issued in spring 2003, may require some water utilities that do not use disinfection to begin doing so. This rule will mainly affect small water utilities, because larger ones have been required to disinfect by previous regulations.

A common method of disinfection is to add chlorine to the water. Out of concern for staying in compliance with the corrosion control stipulations of the Lead and Copper Rule (LCR) (USEPA, 1991), a study was performed to investigate the corrosive effects of chlorine in drinking water systems.

The main goal of this study was to characterize the effect of chlorine on corrosion of metals common to drinking water systems. To accomplish this, pipe loop apparatuses were constructed. Using water that had stagnated in contact with the pipe loops, the concentrations of metals that had leached into untreated water were compared with those in chlorinated water.

A secondary goal was to test whether popular corrosion control chemicals can counteract the possible corrosive effects of chlorine. Metal concentrations from pipe loops with chlorinated water were compared with those from loops with chlorinated water plus a corrosion control chemical.

The experiment was designed to control and compare important factors affecting the corrosivity of water. The two test sites and the chemical additions at each site were chosen to match the factors and characteristics deemed important in the design of the experiment. Comparable test conditions were created at both sites using similar pipe loop apparatuses. Water samples were taken weekly during the yearlong study and were analyzed for water quality parameters and metal concentrations.

A special sampling was performed in which data were gathered to plot as stagnation curves—representations of metal and oxidant levels during the water stagnation period.

Data were analyzed using the Wilcoxon statistical method suggested by USEPA (Wysock et al., 1995). An alternative linear regression method was used to double-check the conclusions. Finally, operational parameters were studied graphically to see whether there was an observable influence on the outcome of the experiments by extraneous factors.

EXPERIMENT DESIGN

Past research has shown that alkalinity and its related parameter, dissolved inorganic carbonate, are water quality parameters that greatly influence the corrosivity of water (EES, 1990; Sheiham & Jackson, 1981).

For that reason, two sites were chosen in this project that differed in these concentrations.

At each site, it was desired to test the corrosivity of the untreated water and compare it with the corrosivity of the same water with chlorine added at the required disinfection dosage.

Three metals representative of typical residential plumbing materials—lead, copper, and galvanized iron—were chosen as the apparatus materials of construction. Although lead piping has been banned in modern drinking water systems, it remains in use in some older systems. In addition, some older systems contain lead solder.

Finally, it was desired to demonstrate corrosion control techniques. Available corrosion control techniques include pH adjustment, alkalinity adjustment, and addition of corrosion inhibitors (EES, 1990). For hard, alkaline water, pH and alkalinity adjustment is not an option because excessive precipitation of calcium carbonate ($CaCO_3$) can occur and diminish the hydraulic capacity of the pipes (EES, 1990). Operators with this type of system water typically add a corrosion inhibitor.

Much success has been found with the use of orthophosphate as a corrosion control chemical for hard, alkaline water (Schock & Fox, 2001; Cantor et al., 2000a; Rezania & Anderl, 1997; Colling et al., 1992). An orthophosphate product was selected and added into chlorinated water at the site with higher alkalinity. Potassium orthophosphate was selected because, with its neutral pH, it is a safer chemical to use than orthophosphoric acid.

Orthophosphate is also effective in lower-alkalinity water. However, in the interest of demonstrating a second corrosion control technique, pH adjustment using sodium hydroxide (NaOH) was chosen and added into chlorinated water at the site with lower alkalinity.

Two test sites were chosen. At each site, untreated water was compared with chlorinated water, and with chlorinated water with a corrosion control chemical added. Each of these scenarios was tested on lead, copper, and iron pipe. There were a total of 18 pipes and conditions.

SITE SELECTION

As stated previously, two sites with different alkalinity concentrations were chosen for this study. The site with hard, alkaline water was the village of Dane (population 620 people) located 20 mi (32 km) northwest of Madison, Wis. The project apparatus drew water from the village's distribution system, which is a mixture of water from two wells. The utility does not chlorinate or fluoridate the water.

The site with softer, less alkaline water was the village of Lone Rock (population 630 people) located 50 mi (80 km) west of Madison. Its distribution system is composed of two wells, but the project apparatus drew water from Well No. 2 only. The pump for Well No. 2 is set to turn on every evening at 10 p.m. and run for about 2 hr until the water tower is filled. The apparatus was set to turn on at 10:15 p.m. and run for 1 hr, ensuring that water flowed through the apparatus only when the well pump was running. The utility does not chlorinate but does fluoridate the water. The apparatus was tapped into the well pump discharge line upstream from where fluoride is added.

Table 1 lists the average water quality characteristics at the two sites. The test waters were not ideal for this study in two ways. First, the tests would have been more demonstrative if there had been a larger difference between the alkalinities of the two sites. Second, both sites exhibited high dissolved oxygen concentrations. The tests would have been more demonstrative with regard to the effect of chlorine as an oxidant if there had not been a high level of dissolved oxygen in the background.

SELECTION OF CHEMICALS

For the chlorinated water in the experiment, a chlorine residual of 0.2 mg/L was achieved. This dosage was based on the requirements of the proposed GWR (USEPA, 2000). The chlorine used was in the form of sodium hypochlorite with 12.5% available chlorine and a density of 10 lb/gal (0.4 kg/L).

For corrosion control at the village of Dane site, a dosage of orthophosphate of 1.0 mg/L phosphorus (P) was achieved based on optimum dosages discussed in the literature (Colling et al., 1992; Gregory & Jackson, 1984). The orthophosphate used was in the form of potassium orthophosphate with 27% total phosphate and a density of 12.7 lb/gal (5.76 kg/L).

For corrosion control at the village of Lone Rock site, 50% NaOH was diluted with deionized water and fed into Lone Rock's water in the apparatus until the pH was adjusted from about 7.7 to 8.2. This was the largest pH adjustment that could be made before $CaCO_3$ began to precipitate into the apparatus.

Table 1 Average water quality characteristics at the two sites

Analysis*	Dane, Wis.	Lone Rock, Wis.
pH	7.4	7.8
Temperature (°C [°F])	14 [57.2]	14 [57.2]
Residual chlorine (mg/L) as Cl_2	< 0.02	< 0.02
Total P (mg/L) as P	< 0.01	< 0.01
Dissolved oxygen (mg/L)	11	6
Total dissolved solids (mg/L)	470	210
Ca (mg/L as Ca)	80	40
Mg (mg/L as Mg)	40	20
Alkalinity (mg/L as calcium carbonate)	290	140
Chloride (mg/L)	60	9
Sulfates (mg/L)	30	15
Manganese (µg/L)	1	3
Lead (µg/L)	1	1
Copper (µg/L)	2	1
Iron (µg/L)	5	2

* Cl_2: chlorine, P: phosphorus, Ca: calcium, Mg: magnesium

DESIGN, INSTALLATION, AND STARTUP OF APPARATUS

The apparatuses were designed similarly to the AWWA Research Foundation (AwwaRF) Pipe Loop Model (EES, 1990). The AwwaRF model is intended to simulate the plumbing of a residence. An important characteristic of pipe loops is that similar test environments can be created so that comparisons of relative corrosivity can be made with the sampling results. Plus, adherence to or divergence from the LCR requirements for corrosion control can be demonstrated.

One difference between the AwwaRF model and this project's apparatuses is that the model is designed for loops that can hold a liter of sample. This project used loops that held a little more than 250 mL of sample. The smaller loops were more economical and took up less space in this situation, in which nine loops were installed at each site.

Other differences with the AwwaRF model involve the operating parameters of flow, pressure, and stagnation time. Because of site constraints, the flow in this project was 1.0 gpm (0.06 L/s) per loop and 60 gpd (227 L/d) per loop. The static pressure was 60–90 psig (0.41–0.62 MPa), and the dynamic pressure was 15–45 psig (0.1–0.31 MPa). The flow and dynamic pressure values reached the lower end of the range that is seen in residential plumbing. There was also great variability of pressures throughout the experiment.

Because the apparatus at the Lone Rock site could not operate unless the well pump was running, the experiment was restricted to operating only once for 1 hr a day. This implies that the water stagnation time in this project was 23 hr. Typically, a pipe loop would be operated over a number of on and off periods throughout the day, as is seen in a residence.

The apparatus at the village of Dane was installed in a heated maintenance building and garage. The building's 0.75-in. (20-mm) galvanized-iron water line running across the ceiling was teed off to supply water to the apparatus. The waste from the apparatus ran a short distance across the floor to a floor drain.

The apparatus at the village of Lone Rock was installed in the Well No. 2 pump house. The apparatus was tapped into an existing 0.75-in. (20-mm) copper pipe that in turn was tapped into the discharge line from the well pump. The waste from the apparatus was piped to a floor drain.

EXPERIMENTAL METHOD

Routine Sampling

Table 2 describes the sample taps on each apparatus. Sample tap locations are shown in Figure 1, a schematic of the pipe loop apparatus.

Samples taken from the sample taps during the study fulfilled one of three goals: (1) to document influent water quality, (2) to document operational parameters and achieve process control, or (3) to document the loop effluent metal concentrations.

The analyses performed on samples from each sample tap are shown in Table 3. For each sampling tap and each parameter, one analysis was performed at the prescribed time of either weekly or quarterly. Water samples were collected and analyzed according to the Code of Federal Regulations (USEPA, 1975) and *Standard Methods* (1995). Table 4 shows the precision achieved for the metal analyses based on duplicate

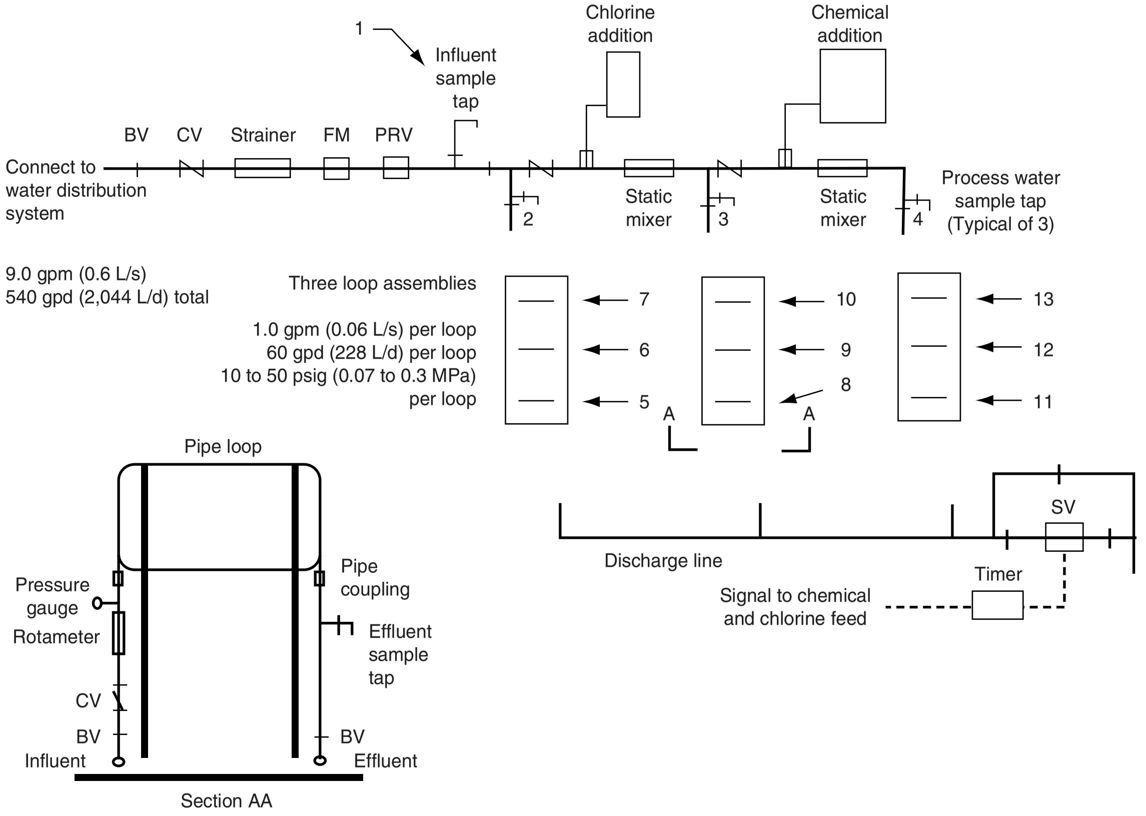
Table 2 Sample taps per site

Sample Tap Number	Description
1	Influent water to the apparatus
2	Raw process water to the first pipe loop group
3	Chlorinated process water to the second pipe loop group
4	Treated chlorinated process water to the third pipe loop group—at Dane, orthophosphate was added; at Lone Rock, sodium hydroxide was added
5	Lead loop using raw process water
6	Copper loop using raw process water
7	Iron loop using raw process water
8	Lead loop using chlorinated process water
9	Copper loop using chlorinated process water
10	Iron loop using chlorinated process water
11	Lead loop using treated chlorinated process water
12	Copper loop using treated chlorinated process water
13	Iron loop using treated chlorinated process water

measurements on 10% of the samples. Details of sampling and analytical methods can be found in the original research article (Cantor et al., 2000b). The parameters pH, temperature, dissolved oxygen, and chlorine residual were analyzed immediately on-site because of their variable nature.

Stagnation Curves

In addition to the described samples, a special sampling was performed in which data were collected for plotting stagnation curves. Stagnation curves are measures of metal concentrations and oxidants (dissolved oxygen, chlorine residual) over various times that the water is in contact with the pipe loop metal (Lytle & Schock, 2000). The curves show that maximum metal concentrations can occur sooner or later than expected. The drop in metal concentration occurs when oxidants are depleted during stagnation. With a high-dissolved-oxygen concentration such as in the water tested in this study, the effect may not be expressed. If the



Numbered items on schematic refer to sample taps described in Table 2. BV: ball valve, CV: check valve, FM: flowmeter, PRV: pressure-reducing/control valve, SV: solenoid valve

Figure 1 Schematic of apparatus

Table 3 Analyses performed on samples from each tap

	Sample Tap Number												
Analysis	1	2	3	4	5	6	7	8	9	10	11	12	13
pH	X[*]	X	X	X	X	X	X	X	X	X	X	X	X
Temperature	X	X	X	X	X	X	X	X	X	X	X	X	X
Residual chlorine	XX[†]	X	X	X									
Total phosphorus	XX	X	X	X									
Dissolved oxygen	XX												
Total dissolved solids	XX	XX	XX	XX									
Calcium	XX												
Magnesium	XX												
Alkalinity	X	X	X	X									
Chloride	XX												
Sulfates	XX												
Manganese	XX												
Lead	X				X			X			X		
Copper	X					X			X			X	
Iron	X						X			X			X

[*] X: samples were taken weekly

[†] XX: samples were taken quarterly

Table 4 Precision of metal analyses

Metal	Concentration Range (*µg/L*)	Precision Based on Duplicate Analyses (%)
Copper	2–10	5
	200–500	2
	501–1,050	1
Iron	2–9	10
	10–40	14
	41–60	3
	100–200	4
Lead	10–40	3
	80–200	6
	201–300	9
	301–900	2

effect does occur, the maximum metal concentration may not be captured during routine sampling, and conclusions may be different.

The curves developed in this study are applicable to the loop conditions around weeks 41 to 45 of the experiment. In addition to measuring metal concentrations during this time, chlorine residual, dissolved oxygen, pH, and temperature were obtained.

Statistical Analysis

Pipe loop data cannot be approximated by a normal distribution. Associated normal distribution methods are therefore not valid. USEPA recommends Wilcoxon nonparametric methods for analysis (Wysock et al., 1995). However, the Wilcoxon methods assume that the samples are obtained randomly. Random sampling implies that the variables that generate the observed values are statistically independent. For data taken in time order, as in this study, correlations over time must be considered, and statistical independence cannot be assumed. When the random sampling hypothesis is inappropriate, erroneous results can occur (Box et al., 1978). To work around this problem, USEPA recommends that only data from pipe loops that have achieved a steady state should be used with the Wilcoxon methods.

Most studies cannot be conducted long enough to reach a steady state. In many cases, it is difficult to prove that steady state has been reached, and some metals, such as copper, can take years to approach a steady state. Therefore, the statisticians involved in this project[*] used an alternative method to compare with and confirm the results of the Wilcoxon method. They elected to study the entire data record and summarize the outcomes of the tests with linear models. That is, a linear equation was derived from the data that showed the separate influence of each treatment scenario plus the effect of time. To do this, a program from specialized software[†] was implemented. The results of the linear model were compared with the results of the Wilcoxon test using the last 12 weeks of data chosen to represent steady-state conditions.

Also in consideration for this study, many factors in this type of experiment cannot be controlled. For instance, the two sites differ in air temperature around the apparatuses as well as system static and dynamic pressures. There are constituents in the water at the two sites that cannot be compared or held steady. For this reason, data on many additional factors were recorded beyond the primary experimental variables. Later, graphing techniques were applied to the data to observe a given factor's influence. Details of the complete statistical data analysis can be found in the research report (Cantor et al., 2000b).

RESULTS AND DISCUSSION

Lead

Figure 2 compares the corrosivity of untreated water on lead pipes, versus chlorinated water, versus chlorinated water with orthophosphate using the high-hardness, high-alkalinity water of the village of Dane. The relative corrosivity is depicted here by graphical means in which the concentration of metal that leaches from a pipe wall into water of specific characteristics is plotted over time. Metal concentrations for water having other characteristics are plotted on the same graph for comparison. In

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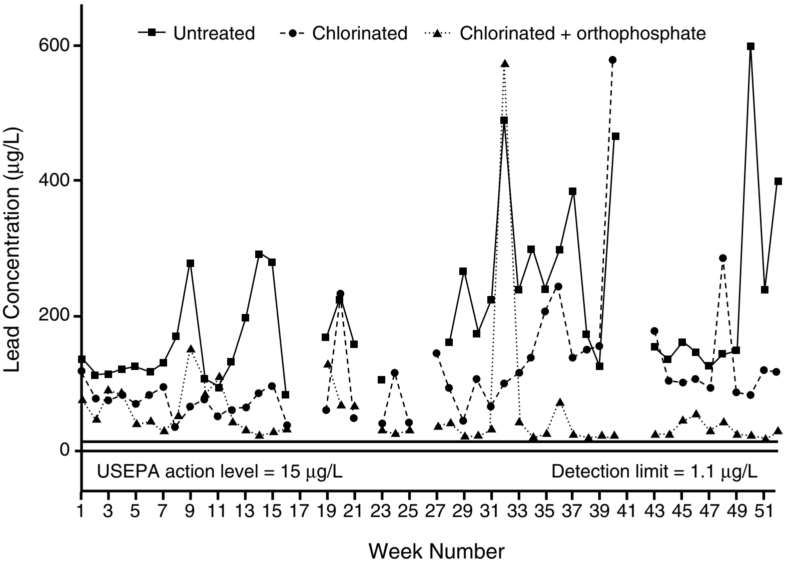
[*] Kevin Little, Informing Ecological Design LLC, Madison, Wis.; Kevin Gross, University of Wisconsin, Madison

[†] Generalized least squares fit routine, S-Plus 2000, Insightful Corporation, http://www.insightful.com



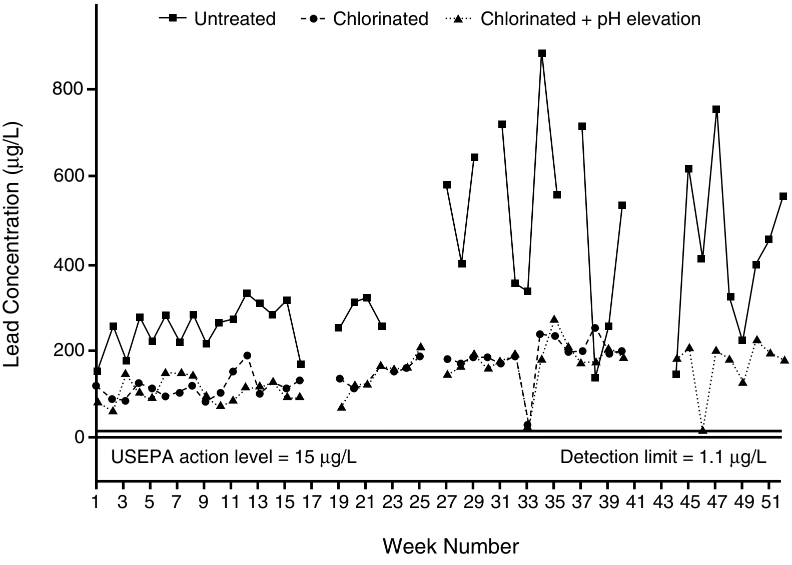
USEPA: US Environmental Protection Agency

Figure 2 Dane, Wis., lead pipe loops

addition, the lead concentrations from the pipe loops can be compared with the health action level (AL) concentration of 15 µg/L of lead set by USEPA (1991).

The lead concentration of the influent water before contact with the lead pipes was below the detection limit of the analytical instrument. The lead concentrations of the open and closed trip blanks were also below the detection limit; therefore, the samples were not contaminated by lead from other sources. The lead in the three types of water under study came solely from the waters' contact with the lead pipes. The USEPA AL appears in the figure as a horizontal line. Lead concentrations above this line are of health concern. Lead concentrations in all three water types were above the AL, but the water with phosphate added dropped to the AL by the end of a year of operation. The corrosivity of the chlorinated water was lower than or equal to that of untreated water. The addition of orthophosphate lowered the corrosivity even further.

In a similar manner, Figure 3 compares the corrosivity of untreated water on lead pipes, versus chlorinated water, versus chlorinated water with pH elevation using the softer, lower-alkalinity water of the village of Lone Rock. Here the chlorinated water also reduced the corrosivity



USEPA: US Environmental Protection Agency

Figure 3 Lone Rock, Wis., lead pipe loops

compared with the untreated water. Elevation of pH did not appear to add further benefit. The lead levels did not drop down as low as the AL in this scenario.

The drop in lead concentrations in the presence of chlorine is addressed in the book *Internal Corrosion of Water Distribution Systems* (AwwaRF & DVGW, 1996). Here, it is mentioned that the "chlorine species could alter lead solubility by oxidizing Pb(II) to Pb(IV) under some conditions." In that case, the solid, lead oxide, is formed and precipitates out, lowering soluble lead levels.

Past research by Lin et al. (1997) has found that water containing oxidants such as dissolved oxygen and free chlorine is the most corrosive to lead. In that research, static leaching tests were performed on lead coupons and copper–lead solder coupons with low-alkalinity waters (31–103 mg/L as $CaCO_3$). These tests compared the effects of free chlorine at a concentration of 1.9 mg/L versus combined chlorine. No comparison was made to untreated water. Water with free chlorine had higher concentrations of lead than water with combined chlorine. In addition, a significant benefit to increasing the pH of the water in order to lower the lead levels was shown. In agreement with the current study, chlorinated water with the higher alkalinity in the study by Lin et al. (1997) showed lower concentrations of lead than chlorinated lower-alkalinity water. Finally, phosphate was added in various doses using various commercial products in Lin et al.'s study in which lower lead levels were achieved in all cases after phosphate was added.

Another research project is difficult to compare but interesting in the trends that were uncovered (Reiber, 1991). In that project, electrochemical activity was measured in a special recycled-flow cell constructed of copper-and-lead-solder–coated pipe in contact with water of low alkalinity. Water at both pH 7 and 9 had similar rates of passivation. When the pH was 6 and 5, the passivation was greatly reduced; that is, the corrosivity of the water increased. An additional experiment in the project looked at the effect of free chlorine. The chlorine residual was increased in increments of 0–1 mg/L and then 1–3 mg/L. At each increment, galvanic current peaked greatly, but this dissipated almost immediately, and the current returned to pre–chlorine addition levels. The researcher concluded that chlorine residuals are of little importance to the galvanic corrosion process. Finally, orthophosphate at a dosage of 0.33 mg/L as P substantially reduced the galvanic current at pH 7. For water of lower pH, the orthophosphate did not have any apparent effect. It was also noted that if the pH of the water dropped below 7, the benefits from a previously formed orthophosphate film were destroyed.

Copper

Figure 4 presents the results from the copper loops at Dane. In general, the three water types produced copper concentrations below the AL of 1,300 µg/L. Chlorine appeared to increase the corrosivity of water in contact with copper. The addition of the orthophosphate further increased the corrosivity by the end of a year of operation, whereas it was hoped that it would counteract the effect of the chlorine.

Figure 5 shows the results from the copper loops at Lone Rock. By the end of a year of operation, chlorine had slightly elevated the corrosivity of the water. The increase of pH intended to counteract the effect of the chlorine had instead increased the corrosivity even more.

Studies have shown that copper corrosion is greatly influenced by environmental factors—pH, temperature, and bicarbonate concentration—as well as concentrations of dissolved oxygen, chloride, nitrate, sulfate, and chlorine residual (Reda & Alhajji, 1996; Edwards et al., 1994; Fujii, 1988).

An experiment by Atlas et al. (1982) found that free chlorine is primarily responsible for the corrosion of copper in chlorinated drinking

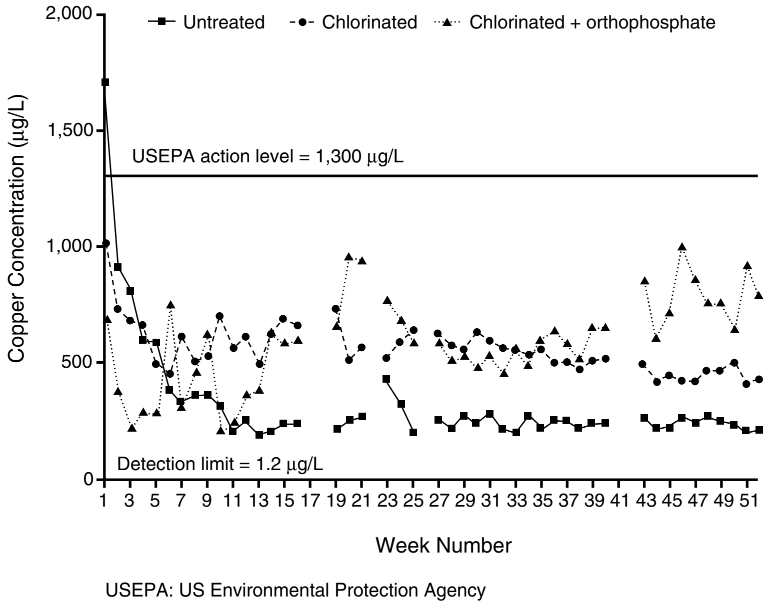


Figure 4 Dane, Wis., copper pipe loops

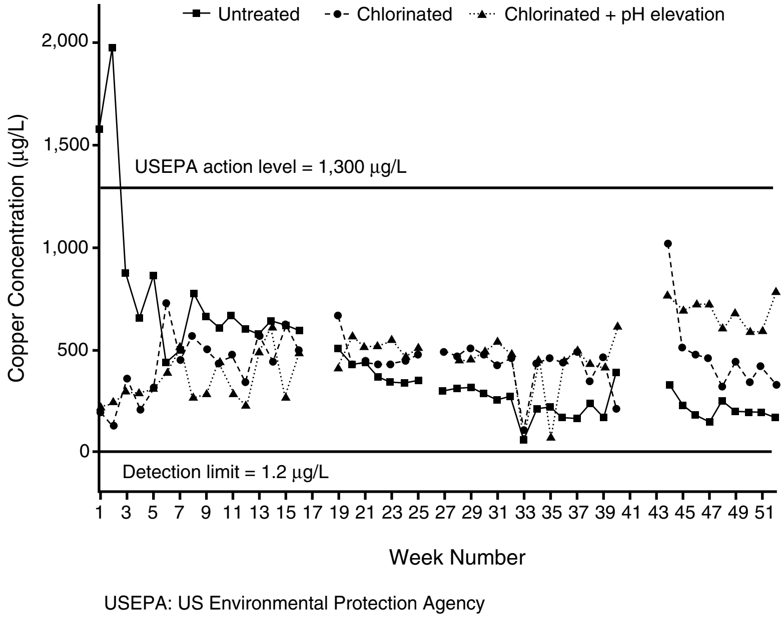


Figure 5 Lone Rock, Wis., copper pipe loops

water systems. Dissolved oxygen, in comparison, plays a relatively minor role. The corrosivity of chlorinated water is enhanced by low pH because of the greater oxidizing strength of hypochlorous acid (favored at low pH) over that of hypochlorite ion. The researchers concluded that free chlorine levels should be maintained no higher than 2 mg/L, and the pH of the water should be maintained between 7 and 8 in order to minimize copper corrosion.

The benefit of adding orthophosphate to decrease the corrosion of copper has been ambiguous in previous experiments as well as the current one (Cantor et al., 2000a). A recent project discussion acknowledged that orthophosphate addition speeds compliance with the drinking water AL for copper in new copper piping at low pH. However, at higher pH levels and over long periods of time, the presence of orthophosphate may stabilize higher copper levels than would naturally evolve (Schock et al., 2000). This seems to agree with the current results.

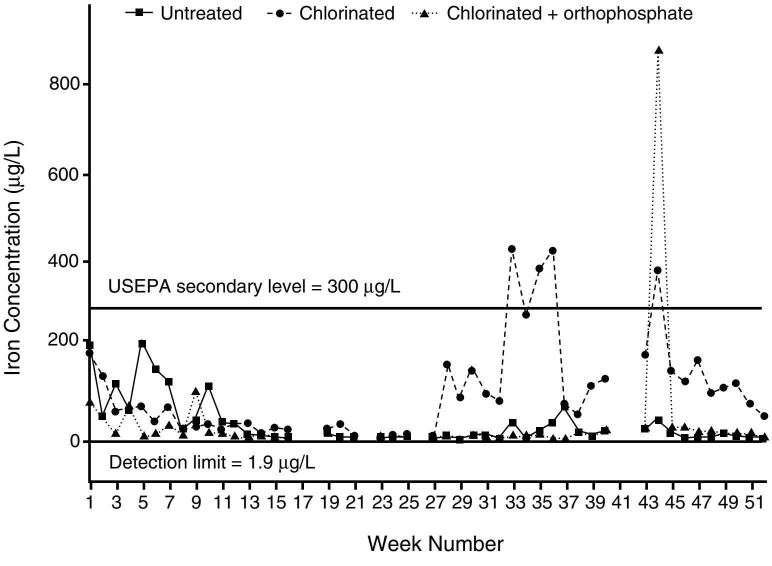
Iron

Figure 6 shows the results from the iron loops at Dane. For iron, USEPA (1979) has set a secondary maximum contaminant level (SMCL) of 0.3 mg/L, above which aesthetic problems such as staining of sinks and laundry occurs but consumer health is not affected. As shown in Figure 6, chlorine increased the corrosivity of water in contact with iron. The addition of orthophosphate brought the iron concentrations back down to the level of the untreated water.

Figure 7 shows the results from the iron loops at Lone Rock. Here, chlorine greatly increased the corrosivity of water in contact with iron. Elevating the pH brought the metal concentrations back down to the level of the untreated water.

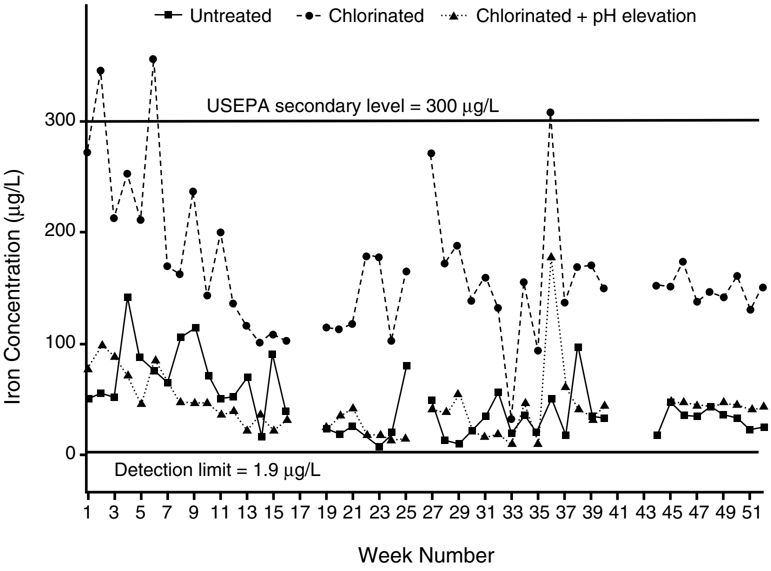
A previous study in which deionized water (water with extremely low alkalinity and hardness) remained at pH 7 in contact with iron pipe showed that a 4-mg/L free chlorine residual was much more corrosive than an equivalent amount of monochloramine residual as a disinfectant (LeChevallier et al., 1990). No comparison was made with untreated water.

The previous research also pointed out that there is a complex circle of events in which free chlorine may cause corrosion and corrosion by-product buildup on the pipe wall. This, in turn, may provide an environment on the pipe wall for biofilm growth. In iron pipes, the corrosion by-products consume the free chlorine before it can penetrate the biofilm to destroy the microorganisms (LeChevallier et al., 1990). With the reduced efficiency of the free chlorine as a disinfectant, higher



USEPA: US Environmental Protection Agency

Figure 6 Dane, Wis., iron pipe loops



USEPA: US Environmental Protection Agency

Figure 7 Lone Rock, Wis., iron pipe loops doses of free chlorine might be used, which in turn would increase corrosion.

To break the circle, LeChevallier et al. (1990) found that implementing corrosion control techniques increased the efficiency of the free chlorine as a disinfectant. The researchers were most impressed with the addition of a polyphosphate blend for corrosion control. Using this chemical, they held the corrosion to a very low rate and found a biofilm inactivation of 2.0-log reduction of viable plate counts versus <0.5 for the same chlorinated water without polyphosphate addition.

Other researchers do not like the idea of adding P in any form to a water system. In some experiments, P addition was observed to consistently stimulate microbial counts (Haas, 1999).

If a disinfectant is not added to the water, a possibility exists for growth of microorganisms in the system. The presence of microorganisms can cause increased corrosion in a water system (AwwaRF & DVGW, 1996). The remedy for this calls for the addition of a disinfectant. In this case, corrosion will be reduced by the addition of chlorine.

The previous research projects concerning iron point to a very delicate balance of pH, free chlorine levels, and possible phosphate levels to control corrosion while effectively controlling microorganism growth in the water system.

Stagnation Curves

Stagnation curves for lead, copper, and iron at the two sites were developed after the 41st week of the experiment. In some cases, a maximum metal concentration was reached before 23 hr of stagnation time. This means that the true corrosive effect of the water was not measured during routine sampling. However, the relative positions of the untreated, chlorinated, and chlorinated/corrosion control chemical addition data curves on the graph remained the same. Therefore, previous conclusions concerning relative corrosivity were unchanged.

One exception is shown in Figure 8. In the lead loops at Dane, chlorinated water exhibited greatly higher lead levels than the untreated water initially. After 6 hr of stagnation, however, these lead levels approached each other at around 150 µg/L. The chlorinated water had probably precipitated lead compounds that do not go back into solution as oxidant levels, pH, and temperature change over time. During the stagnation curve period, the chlorine residual in the two lead loops with chlorinated water stayed constant at about 0.13 mg/L for 9 hr of stagnation and then dropped to 0.03 mg/L by the end of 23 hr. The dissolved oxygen was high in the system, at around 9 mg/L, and dropped

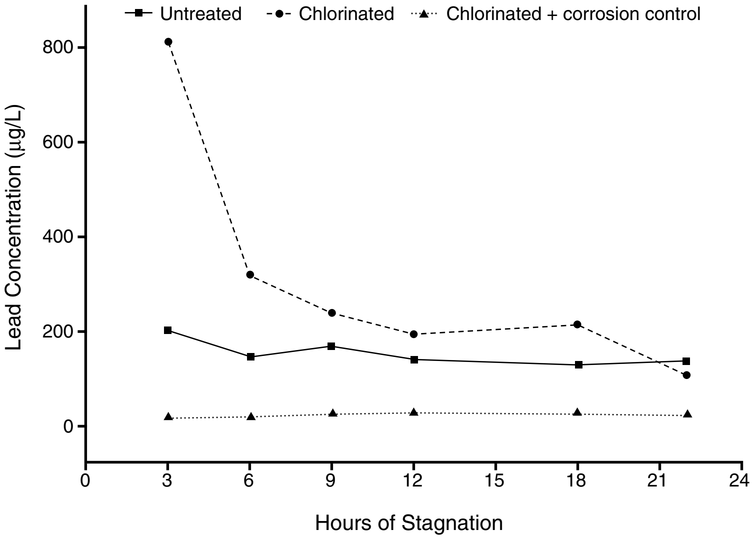


Figure 8 Dane, Wis., stagnation curve for lead

only slightly to 7 and 8 mg/L after 23 hr. Temperature increased as the water stagnated for 23 hr and warmed to room temperature. The pH dipped from 7.5 to 7.25 after 12 hr and then increased back to around 7.35 at the end of 23 hr.

Determining the Effectiveness of Treatments

The Wilcoxon statistical analysis of the data confirmed the observations from the graphs discussed previously. In addition, the linear model did not change the conclusions from the Wilcoxon test. However, the linear model did add information about the increase or decrease of treatment effects over time. It also showed the magnitude of one treatment effect compared with another. This information can also be confirmed by simple observation of the data graphs, except that the statistical significance of an observed effect can be quantified.

Studying the Influence of Other Factors

A second goal in analyzing the data was to view the influence of various water quality and operating parameters on the results of the study. Many such factors cannot be controlled in on-site pipe loop studies, as they should be in a more rigorous scientific experiment.

The influence of various factors was determined qualitatively by graphical means. The operating and water quality parameters did not appear to greatly influence the outcomes of the tests. Most of these parameters suggest that the site location (higher-alkalinity versus lower-alkalinity water) and the treatment (untreated, versus chlorinated, versus chlorinated with corrosion control chemicals) are the main influences, as was hoped in the experimental design. Temperature does not affect the lead and copper levels much, but greatly affects the iron levels. Outcomes appear to be independent of pressures and flows.

CONCLUSIONS

The goal of this study was to observe the effects of chlorine addition on the corrosivity of water. The water available for study was high in dissolved oxygen content. The results of this study therefore cannot represent conditions in which no significant quantities of oxidants exist in the water before chlorine is added. The results for the water with high dissolved oxygen content are summarized in the following material.

Iron appears to be the most affected by free chlorine addition, followed by copper, followed by lead, which may or may not experience increased corrosion.

Copper concentrations in the untreated water at both sites were below the USEPA AL of 1,300 µg/L. The addition of chlorine increased the concentrations. However, the increased concentrations did not exceed the AL. In a similar manner, iron concentrations elevated by chlorine at both sites typically did not exceed the SMCL of 0.3 mg/L. Lead concentrations at both sites were above the USEPA AL of 15 µg/L in the untreated water, so predictions on the effect of chlorine cannot be made for systems already in compliance with the LCR.

In this study, elevating the pH of the water at the village of Lone Rock site, originally with a pH of 7.8 and a total alkalinity of 140 mg/L as $CaCO_3$, was beneficial in counteracting the corrosive effect of chlorinated water on iron. However, the elevation of pH was not beneficial in controlling corrosion of lead or copper.

Adding orthophosphate decreased corrosion of lead and iron in contact with chlorinated water at the village of Dane site (pH of 7.4 and a total alkalinity of 290 mg/L as $CaCO_3$). However, copper corrosion appeared to reach an increased level in the long term.

RECOMMENDATIONS

If elevated levels of metals become a problem after chlorine addition, a first step in counteracting the increased corrosion should be to elevate the pH of the water, but great care must be taken. A pH above about 7.8–8.0 greatly diminishes the effectiveness of chlorine as a disinfectant (Connell, 1996). A pH below 7 creates water of high corrosivity (EES, 1990; Sheiham & Jackson, 1981). In water with high alkalinity and high hardness, an increase in pH may cause excessive precipitation of $CaCO_3$ with associated hydraulic problems in the piping system. It has been suggested that the $CaCO_3$ precipitation potential (CCPP) be within the range of 3–10 mg/L (RTW Professional Engineers, 1994). Therefore, if the water characteristics allow an increased pH with a CCPP in the proper range and the alkalinity of the water is at moderate to low levels below 200 mg/L as $CaCO_3$, then the pH should be elevated to between 7 and 7.8.

If one or more metals in the drinking water system continue to show increased corrosion levels, or if pH elevation is not practical in the first place, orthophosphate should be added to the chlorinated water. The pH of the water should be at a minimum of 7 before orthophosphate is added. Past research has established an effective dose of orthophosphate to be 1 mg/L as P, or higher initially, later being reduced to an effective maintenance dosage (Colling et al., 1992; Gregory & Jackson, 1984; Sheiham & Jackson, 1981). The operator must also be aware that phosphate addition to drinking water may cause conflicts with the P discharge limits at the wastewater treatment plant and with P levels allowable for runoff into nearby lakes and streams. Also, with orthophosphate addition, copper corrosion in the system may increase. The increase can be tolerated if copper levels do not rise above 1,300 µg/L.

Proper monitoring is essential to proper corrosion control. Chemicals should not be added to a system without appropriate feedback regarding their effectiveness. However, several problems with regard to monitoring exist.

With the monitoring prescribed by the LCR, the emphasis is on selecting older buildings with lead or lead solder in the plumbing systems for sampling. Newer homes with copper plumbing are not included, sometimes causing a false sense of security about copper levels in the system (Cantor et al., 2000a). Other materials of construction, such as iron, which do not cause major health concerns, are also not represented in this sampling technique.

An established and effective monitoring method, focusing on the health and regulatory issue of metal release, is to use pipe loops similar to the ones used in this project. A loop of each metal of interest can be connected at one or more locations in the water distribution system. Routine samples taken from the pipe loops will describe the corrosive trends in the system, and chemicals can be adjusted accordingly. Other advantages of this method are that the conditions under which samples are taken can be better controlled than in a private residence, and the operator does not have to depend on the goodwill of consumers to obtain samples. The disadvantage of this system is that a small water utility especially does not have the resources and personnel to attend to the apparatuses.

Quicker and less expensive methods of monitoring have not yet been established as providing a clear picture of corrosion for various metals in a system. Research is proceeding in this direction.

Until conclusions are established on new, effective, and economical corrosion monitoring techniques, water utility operators who do not use pipe loops for monitoring should obtain samples from buildings in their distribution system in addition to those required for the LCR.

Careful sampling can be achieved by paying attention to the following details:

- Select buildings close to the utility water source and at the extreme ends of the distribution system. Other factors should be taken into consideration for sampling site selection as well. Any situations in which the water chemistry may be affected, such as mixing zones, different pressure zones, and temperature extremes, are important for tracking corrosion.
- Verify the materials of construction in the buildings selected for sampling.
- Verify that the water being sampled has not been softened or treated by any other means at the sample site.
- Verify that no water flowed through the building's plumbing system for the prescribed stagnation time of a first-draw sample.
- Be consistent in the stagnation time of each sample for all sampling events.
- After obtaining the first-draw sample for metals, gather other information, such as the temperature and pH of the stagnated water, the temperature and pH of the flowing water, the

orthophosphate concentration of the flowing water (if using any phosphate product), the total phosphate concentration of the flowing water (if using a polyphosphate product), and the chlorine residual of the flowing water.

- Plot all information on graphs over time. Note changes to corrosion control treatments or other system changes on the graphs.

Only by monitoring system metal concentrations on a routine basis can utility staff track the health of the system in terms of corrosion and react appropriately and in a timely manner to unwanted changes.

ACKNOWLEDGMENTS

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CHAPTER · 24

Effect of Grounding and Electrical Properties on Water Quality

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SUMMARY

The use of metallic water piping as part of a building's electrical grounding system has been a commonplace practice for more than 80 years. When electrical transformers serve multiple buildings, the water services and distribution piping can act as parallel neutral return paths for stray current from building electrical systems to the transformers. Electrical resistance in pipe connections, if present, forces some fraction of the stray current to flow through the parallel water path. Discharge of current on the inside of the pipe increases metal release. Previous studies on the effects of grounding on water quality and metal release did not investigate the effects of resistance or dielectric fitting. This study investigated the effects of dielectric fittings on water quality. Copper tubing that had been modified with inserted dielectric fittings was filled with potable water from two different water sources and was tested with applied alternating current (AC) voltages of up to 123 V for periods of up to 110 hr. Significant increases in copper, lead, and zinc in stagnant potable water were measured in less than 24 hr at applied voltages of ~50 VAC. Metal release generally increased with charge transfer, in accordance with Faraday's law.

INTRODUCTION

The practice of using the water piping as all or part of a structure's grounding electrode system has been commonplace for more than 80 years (Kennedy, 1952). Grounding is necessary to minimize the hazards that can exist when humans interact with electrical equipment and machinery. The general requirements for grounding of electrical equipment and systems are found in article 250 of the National Electrical Code (NEC, 1993).

In order to better understand the consequences of grounding the electrical system of a building to water pipes, the AWWA Research Foundation funded a study to evaluate the effects of grounding on pipe integrity and shock hazard. The results of those investigations have been published (Duranceau et al., 1998; Duranceau et al., 1996). This chapter presents results from one portion of the study, which looked into how grounding affects pipe metal release and water quality.

PREVIOUS INVESTIGATIONS

For many years, the water utility industry has had concerns about the effect of grounding currents on water quality. Early research on the subject included studies that focused primarily on grounding's effects on water quality in terms of taste and odor (American Research Committee on Grounding, 1944; Eliassen & Goldsmith, 1944; Eliassen, 1941; Warren, 1934). In these studies, alternating current and direct current (AC and DC) were conducted along lengths of electrically continuous metallic water service piping and measured the change in metal ion concentration over a period of time. Changes in metal content of the water were measured using the analytical techniques available at the time. These methods had higher limits of detection compared with today's analytical methods. These early studies concluded that the presence of AC and DC on electrically continuous metallic water service piping did not affect water quality.

The promulgation of the Safe Drinking Water Act's Lead and Copper Rule (LCR) in 1991 again raised concerns about the effect of grounding currents on water quality in terms of metal release. A study by the Orlando (Fla.) Utilities Commission was performed using a model house system (OUC & CH2M HILL, 1994). The study concluded that grounding currents flowing on electrically continuous potable water pipes do not cause characteristic and distinct increases in metal concentrations for the conditions investigated. However, posttest examination of the pipe surfaces indicated changes in protective oxide films, suggesting some effects of AC. Further, increases in metal concentrations were measured during and after lightning storms. Similar results regarding effects of lightning on internal corrosion and metal release have been reported in the literature (Guererra, 1979).

Previous studies on the effects of grounding on water quality and metal release have not investigated the effects of resistance or dielectric fittings on water quality. Without significant resistance in the metal pipe wall, electrical current continues to flow in the much more conductive pipe wall (OUC & CH2M HILL, 1994; Guererra, 1979; Eliassen, 1941). Current is not discharged on the inside or the outside of pipe, and it is not surprising that no effect on water quality was previously found when electrically continuous pipe sections were tested. The study described here investigated the effects of dielectric fittings (as simulations of electrical discontinuities in pipe continuity) on water quality in terms of metal release.

PARALLEL PATHS

Many water utilities use plastic service piping, dielectric connections, or both. Further, even if they are not intentionally insulated, rubber-gasketed meter connections can act as resistive or insulating connections in the service.

Figure 1 shows schematic diagrams of current paths for water services. When electrical conditions in the house are such that current is forced to flow on the water service, the current will divide itself among three parallel paths: the soil, the water, and the pipe wall. Figure 1A shows the typical situation for buried electrically continuous water service, Figure 1B shows the current path for buried electrically discontinuous water service, and Figure 1C shows the current path for electrically discontinuous water services in air.

The soil current path is represented by an equivalent circuit composed of the soil double-layer capacitance, $C_{DL,Soil}$, in parallel with the charge transfer or Faradaic resistance, $R_{CTR,Soil}$, both in series with the soil solution resistance, R_{Soil}. $C_{DL,Soil}$ represents the charge separation that occurs across the metal-soil interface. $R_{CTR,Soil}$ is the thermodynamic resistance to charge transfer, and R_{Soil} is related to the resistance through the bulk soil path between the points at which the potential is applied.

A similar equivalent circuit can be defined for the water path on the inside of the tube. The resistance of the metal wall is simply related to the metal tube geometry (cross-sectional area of the wall and overall length) and the resistivity of the metal. Current flowing through the soil circuit results in external corrosion. Current flowing in the internal or water circuit results in metal release. Current flowing through the wall heats the metal but does not result in corrosion, either externally or internally.

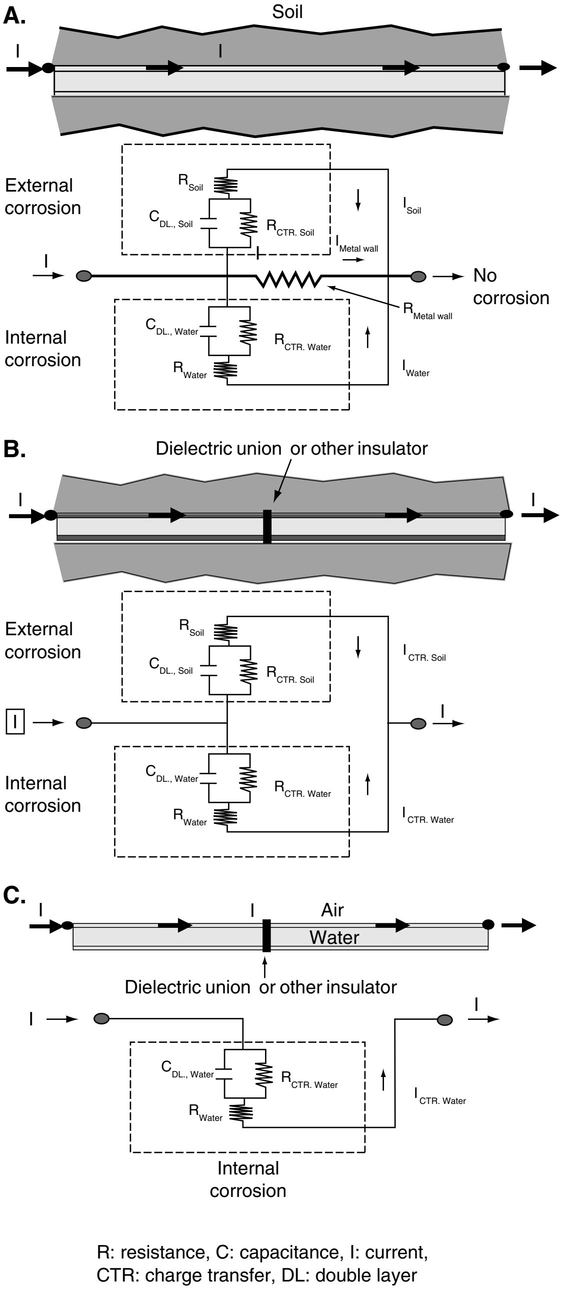

Figure 1 Schematic diagram of current paths for buried electrically continuous water services (A), buried electrically discontinuous water services (B), and electrically discontinuous water services in air (C)

Because the conductivity of most metal service line materials is much greater than that of soils and waters by many orders of magnitude, the vast majority of the current flows along the wall and does not produce internal or external corrosion.

Figures 1B and 1C show the situation when a dielectric or insulator is present, either intentionally or unintentionally, in the water service in soil or in air. For the case of a buried insulator or resistor in the water service (Figure 1B), currents are forced to flow in both the soil and the water paths. Both internal and external corrosion can occur. For the case in which the insulator occurs above ground (Figure 1C), e.g., in a meter box or basement, current will only flow in the water path, causing internal corrosion and metal release.

Electrically insulating or resistive fittings can create anodic and cathodic areas on each side of the dielectric union when currents (even small amounts) flow through the parallel high-resistance water path. The discharge of current on the interior of the pipe can increase the metal content of the water. Various utilities had verbally provided information about their experiences with degraded water quality caused by grounding currents (Duranceau et al., 1996). The LCR requires water treatment as a method for corrosion control and reduction of metal concentrations at the customer's tap if the concentration of lead (Pb) or copper (Cu) exceeds specified limits. However, if dielectrics or electrical discontinuities in the piping system contribute to metal release, it is possible that water treatment will not solve the problem. As part of a larger study (Duranceau et al., 1996), two sets of tests investigated the effects of dielectric fittings on water quality in terms of metal release.

INITIAL STUDIES

Initial Experimental Plan

For more than 40 years, the East Bay Municipal Utility District (EBMUD) in Oakland, Calif., has had policies and construction procedures that require the installation of electrical insulating materials (e.g., dielectric unions, plastic piping) in service lines. EBMUD developed and implemented a simple experimental plan of laboratory tests that could quickly investigate AC effects on water quality where insulating unions were present in copper service lines.

EBMUD set up the experiment at its Claremont Center Plant using a bench-top pipe rig. The test articles were designed to simulate the situation shown in Figure 1C. Test pipe specimens consisted of two 18-in.

(460-mm) lengths of 0.75-in. (20-mm) type K copper tubing soldered with lead-free solder to each side of a brass insulating union. The test article was filled with approximately 0.25 L of potable water and sealed with rubber stoppers on both ends. AC was supplied to the test pipe specimens via #12 solid copper wire clamped to one end of the test article and the neutral return (#12 solid white) clamped to the other end. The AC applied was measured using a 0.01-ohm shunt on the neutral return side of the test article. Currents and voltages were measured using a true root-mean-square 4.5-digit digital multimeter.*

Tests were run with applied AC voltage levels of 123 volts of AC (VAC) (full-line voltage) and approximately 50 VAC (reduced through a light dimmer). No rectified DC was observed during the experiments. Control specimens consisted of straight runs of copper tubing filled with potable water and stoppered on both ends. No dielectric unions were used for control specimens in the first series of tests, and no voltage was applied to the control specimens. Samples of the water from the test specimens were taken periodically and analyzed for Cu and, in some cases, zinc (Zn), using inductively coupled plasma. Pb was not considered in this first series of tests. Prior to testing, samples were homogenized by inverting the test article several times before sampling. The pH of the water sample was also measured. Results of the two test runs are shown in Table 1.

The first test was run at 123 VAC. Currents ranged from ~47 to 80 mA. The first test ran a total of 1.9 days. One specimen (1A) was removed after 22 hr; the other four (1B, 1C, 1D, and 1E) were removed after 46 hr. The second test was run at approximately 50 VAC. The specimens were removed after 24 hr (2C), 86 hr (2D), and 110 hr (2E). Currents ranged from 14.5 to 25.4 mA AC.

Results of Initial Tests

Cu, Pb, and Zn pickup by the water from the test specimens was significant. These data indicate that AC voltage had a pronounced effect on metal release for both the 123- and 50-VAC tests. Cu and Pb contents measured after 24 hr of exposure at the lower applied voltage exceeded US Environmental Protection Agency (USEPA) action levels (ALs) of 1.3 and 0.015 mg/L for Cu and Pb, respectively.

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* Fluke model 87, Fluke Corp., Everett, Wash.

Table 1 Summary of current and metal concentration data for EBMUD* testing

	Parameter†					
Test Pipe	Average AC *mA*	CT *hr*	AC Voltage Across IU *V-RMS*	Total Cu Concentration *mg/L*	Total Pb Concentration *mg/L*	Total Zn Concentration *mg/L*
Raw water	NA‡	NA	NA	0.003–0.0198	0.003	0.003
Average of five controls	0	24–168	0	0.296	0.030	0.084
1A	47	22	123	7.47	0.248	10.2
1B	56	46	123	15.3	0.476	20.6
1C	67	46	123	19.0	0.718	25.3
1D	51	46	123	5.66	0.346	15.4
1E	80	46	123	40.2	0.141	33.5
2C	21	24	54.5	1.64	0.211	7.04
2D	14.5	86	53.2	1.71	0.549	18.2
2E	25.4	110	49.2	3.52	0.813	15.1

*EBMUD: East Bay Municipal Utility District

†AC: alternating current, CT: contact time, IU: insulating union, RMS: root-mean-square, Cu: copper, Pb: lead, Zn: zinc

‡NA: not applicable

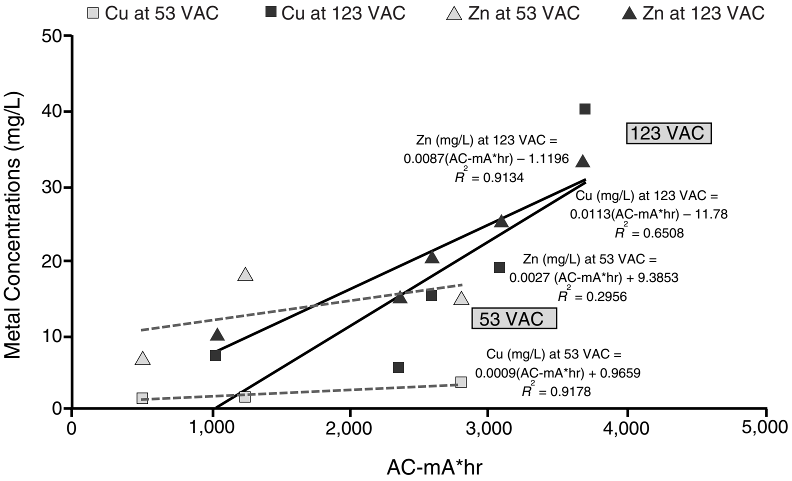
Average metal release caused by the measured AC was approximately 0.13% of that which would be expected from a similar amount of DC. This is considerably lower than the widely used rule of thumb of 1% of DC. This rule may still be valid for metal consumption, as opposed to metal release, because Cu consumed by oxidation reactions may be in the form of insoluble oxides and hydroxides, which would not be measured as metal release.

According to Faraday's law, metal release is related to charge transfer. Figures 2 and 3 show the dependence of metal pickup of the water as a function of the product of the average mA of AC and the contact time (CT), i.e., AC-mA*hr, which is a gross measure of charge transfer. For the 123-VAC tests, concentrations of Cu and Zn increased with AC-mA*hr, whereas Pb first increased and then, near the end of the test, decreased (see Table 1). The reason for the decrease was not apparent. A similar trend occurred with Zn in the ~53-VAC tests, although Pb and Cu concentrations continued to increase with AC-mA*hr. Increases in Zn and Pb concentrations were much larger at ~53 V, compared with the less electrochemically active Cu. These results were consistent with the other findings of AC influence on corrosion rates (Bruckner, 1963).

Luggin-type probe salt bridges to Cu–copper sulfate reference electrodes on each side of the insulating union on one test specimen provided a low-resistance contact to the water inside and allowed measurement of changes in the pipe-to-water (P/W) potential as current was applied. Changes in P/W potentials were indicative of changes in the electrochemical state of the metal in the vicinity of the salt bridge and therefore represented changes in the corrosion behavior of the copper piping. A shift in the negative direction indicated that cathodic reactions were favored and oxidation was reduced and vice versa.

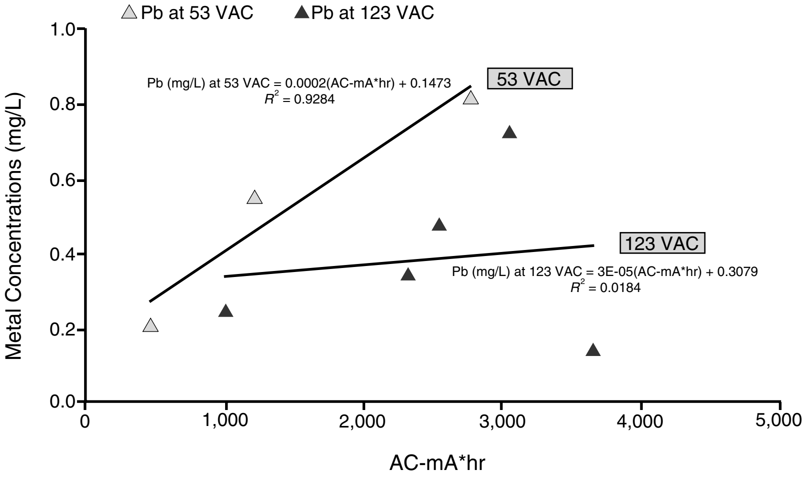
Table 2 shows P/W potential data as a function of CT for specimen 2E. The data indicate that the hot side of the dielectric was cathodic with respect to the neutral side. DC potential difference between the hot and neutral sides increased with CT, whereas the AC potential difference first increased and then decreased.

The pH of the water samples from both tests varied from a low of 7.6 to a high of 9.6. Variations could not be correlated with the test parameters. However, increasing pH to 9.6 did not appear to significantly reduce Cu release. It should be noted that EBMUD potable water at the test site was drawn from snowmelt. It had very low conductivity, with total solids of about 84 mg/L, which relates to an electrical resistivity of about 8,000 ohm-cm. All other factors being equal, similar tests with



Cu: copper, Zn: zinc, AC-mA*hr: metal pickup of the water as a function of the product of the average mA of alternating current and the contact time, EBMUD: East Bay Municipal Utility District, VAC: volts of alternating current

Figure 2 Cu and Zn concentrations as a function of AC-mA*hr for initial scooping tests at EBMUD with 53 and 123 VAC applied



Pb: lead, AC-mA*hr: metal pickup of the water as a function of the product of the average mA of alternating current and the contact time, EBMUD: East Bay Municipal Utility District, VAC: volts of alternating current

Figure 3 Pb concentrations as a function of AC-mA*hr for initial scooping tests at EBMUD with 53 and 123 VAC applied

Table 2 AC and DC P/W potential as a function of CT[*]

CT *hr*	AC Voltage Across Union *V*	P/W Potential Hot Side *mV to CSE*[†]		P/W Potential Neutral Side *mV to CSE*	
		AC	DC	AC	DC
Zero before startup no applied voltage	0	24	+014	27	+042
0.5	52.9	734	–039	139	+006
24	54.5	525	–205	130	–155
88	53.2	146	–293	65	–181
110	49.2	139	–304	64	–139

[*] AC: alternating current, DC: direct current, P/W: pipe-to-water, CT: contact time

[†] CSE: copper sulfate electrode

more conductive waters (see series of tests discussed in subsequent sections) should produce higher currents with higher metal pickup.

FOLLOW-UP STUDY

Study Objectives

The data gathered during the initial scoping study were sufficiently interesting to warrant verification and additional investigation. EBMUD and Pinellas County Utilities (PCU) agreed to participate in a follow-up study. The study had the following objectives:

- Repeat and confirm results from the first series of bench-scale experiments at EBMUD and a different water source (PCU)
- Investigate the effects of short sections of plastic pipe as insulators on water quality
- Evaluate the effect of lead solder surfaces and solder joints on metal (Pb, Cu, and Zn) release

Table 3 compares EBMUD and PCU water quality. Because EBMUD is a surface water and PCU is a groundwater supply, each has different average water qualities. The values shown in Table 3 are general averages and are for comparison purposes only.

Test Conditions and Test Article Design

The follow-up study set out to investigate four voltages (120, 25, 10, and 5 VAC). Three types of test articles (A, B, and C) were used in the testing for three lengths of time (24, 72, and 168 hr). Each type of test article had an associated control with no applied voltage for 168 hr. In addition, each utility fabricated two "spare" test articles (15 of each type—a total of 45 test articles of all three types) and selected 13 test articles for use in the testing. The two untested test articles, if unused, were archived for quality assurance and control purposes.

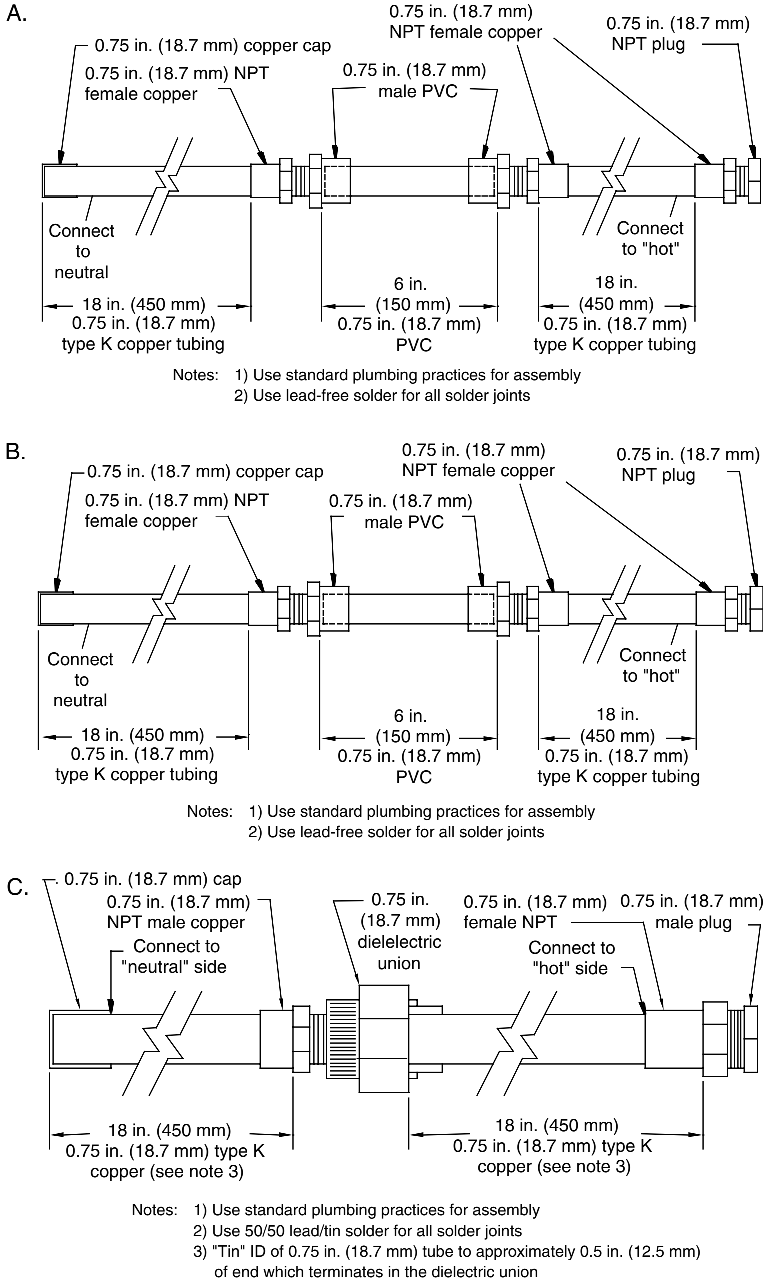
Figure 4 shows the test article design for the experiments conducted with both EBMUD and PCU. Test articles A were the most similar to the tests previously run at EBMUD. Test articles B investigated the use of 6 in. (150 mm) of polyvinyl chloride (PVC) pipe as an insulating method. Test articles C investigated the effects of dielectric unions on systems in which lead solder was used to make joints with copper. Although the design of the test articles used at the two utilities was similar, their fabrication resulted in differences in the electrical properties.

Table 3 General comparison of EBMUD and PCU water quality[*]

Parameter	EBMUD		PCU	
Water source	Surface water		Groundwater	
Corrosion control	pH adjustment		Corrosion inhibitor	
Disinfection	Chloramines (liquid chlorine)		Free chlorine (gas chlorine)	
Water Quality	Raw	Finished	Raw	Finished
pH	7.0–9.2	8.3–9.3	7.3–7.7	7.7–7.8
Temperature (°C)	10–25	10–25	23–25	23–25
Alkalinity (mg/L as $CaCO_3$)[†]	15–140	18–128	192–212	192–212
Total hardness (mg/L as $CaCO_3$)	25–165	12–144	200–242	200–242

[*] EBMUD: East Bay Municipal Utility District; PCU: Pinellas County Utilities

[†] $CaCO_3$: calcium carbonate



A. Notes: 1) Use standard plumbing practices for assembly
2) Use lead-free solder for all solder joints

B. Notes: 1) Use standard plumbing practices for assembly
2) Use lead-free solder for all solder joints

C. Notes: 1) Use standard plumbing practices for assembly
2) Use 50/50 lead/tin solder for all solder joints
3) "Tin" ID of 0.75 in. (18.7 mm) tube to approximately 0.5 in. (12.5 mm) of end which terminates in the dielectric union

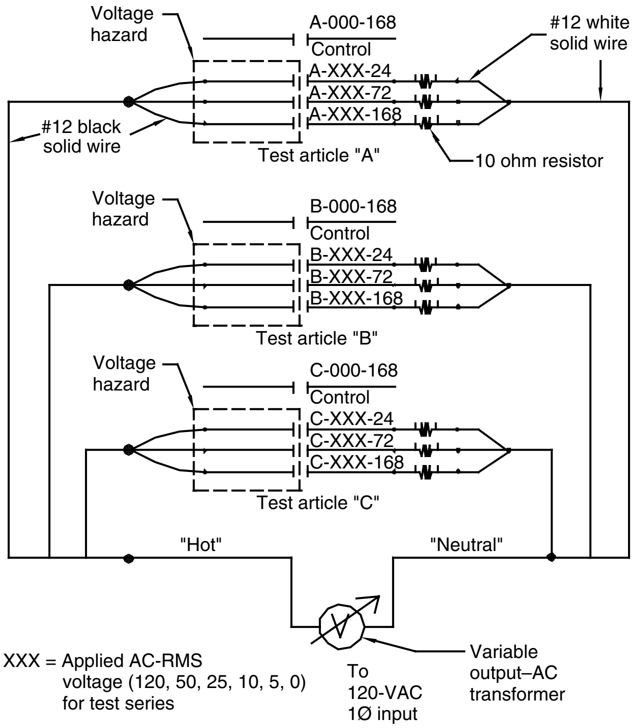
Type A: copper–dielectric union–copper, type B: copper–polyvinyl chloride pipe–copper, type C: copper + lead solder–dielectric union–copper + lead solder, PVC: polyvinyl chloride

Figure 4 Diagrams of test articles used in follow-up testing: type A (A), type B (B), and type C (C)

Figure 5 shows the wiring diagram for the testing. The test articles were wired in parallel so that the applied voltage across all the test articles was equal. A 10-ohm resistor (± 2%) on the neutral side of the test articles acted as a current-measuring shunt for each test article.

Data Collection Procedures

At startup of a test series (i.e., 120, 25, 10, and 5 VAC), AC voltage and 10-ohm resistor voltage drop (10 mV = 1 mA) were measured every 5 min for the first half hour. After this initial startup period, at least three daily measurements (morning, noon, and end of day) were made of AC voltage across insulator or PVC tubing along with current flowing in each 10-ohm resistor. Measurements were made with properly rated test leads and meters by personnel trained in electrical safety and familiar with the hazards particular to these tests.



AC: alternating current, RMS: root-mean-square, VAC: volts of alternating current

Figure 5 Wiring schematic for follow-up testing

RESULTS

Tables 4 and 5 summarize data from the follow-up study for EBMUD and PCU testing, respectively. Tests were identified by the test article, applied voltage, and exposure time. For example, "A-005-024" designates the test conducted with test article type A with 5 VAC applied across the insulator for 24 hr.

Direct comparison of the PCU and EBMUD C series samples was not possible because of differences in fabrication and resulting electrical characteristics for the two utilities' test articles. PCU was unable to perform tests at 120 V because of high currents, which caused shorting across one of the dielectric unions and increased water temperature above 60°C. Measured resistances were approximately four times lower at PCU than at EBMUD, probably because of the higher conductivity of the PCU groundwater compared with the low conductivity of EBMUD snowmelt water. PCU conducted tests with a maximum of 50 VAC applied voltage instead of 120 VAC.

Figures 6–9 show representative examples of the dependence of metal pickup of the water as a function of the product of the average mA of AC and CT (AC-mA*hr) for Cu, Zn, and Pb for each of the test article types for both EBMUD and PCU.

Description of A Series Test Articles

For the A series test articles, which consisted of copper tubing separated by a dielectric union fabricated with lead-free solder, Cu, Zn, and Pb increased with increasing AC-mA*hr for both the EBMUD and PCU data. The level of metal release was lower for PCU than for EBMUD, despite the fact that higher currents were present at PCU compared with those at EBMUD. PCU uses a blended phosphate corrosion inhibitor to control Cu corrosion, whereas EBMUD does not use a corrosion inhibitor. The presence of the corrosion inhibitor probably contributed to the differences in metal release between the two utilities and sets of experiments. The levels of metal release for this second series of tests at 120 V were consistent with the levels found in the initial scoping tests and confirmed the initial results.

Description of B Series Test Articles

Total metal concentrations were substantially lower for both PCU and EBMUD for the B series test articles, which consisted of copper tubing separated by 6 in. (150 mm) of PVC pipe as an insulating connector. The

Table 4 EBMUD water quality data*

	Parameter†								
Test Number	Cu *mg/L*	Pb *mg/L*	Zn *mg/L*	Average Current *AC-mA*	Average Resistance *ohm*	mA*hr	pH	Conductivity *mS/cm*	DO *mg/L*
5 VAC MW‡	<0.003	0.002	0.014	NA§	NA	NA	7.20	57	9.2
10 VAC MW	0.014	0.004	0.026	NA	NA	NA	8.70	63	9.60
25 VAC MW	0.014	0.004	0.026	NA	NA	NA	8.70	63	9.60
120 VAC MW	<0.003	0.002	0.014	NA	NA	NA	7.20	57	9.2
A-000-168	5.900	3.500	0.760	NA	NA	NA	8.50	67	10.0
A-005-024	8.240	0.958	0.537	0.740	6,757	17.8	8.60	67	9.3
A-005-072	5.290	0.276	1.600	0.960	5,208	69.1	9.10	68	9.1
A-005-168	5.400	10.000	3.300	0.860	5,814	144.5	8.90	65	8.0
A-010-024	1.260	0.524	1.780	2.550	3,922	61.2	7.50	65	9.0
A-010-072	1.260	0.394	1.490	2.350	4,255	169.2	8.00	67	7.9
A-010-168	10.500	0.593	1.220	2.280	4,386	383.0	8.80	69	6.9
A-025-024	2.410	0.164	1.840	3.840	6,510	92.2	8.80	62	8.6
A-025-072	13.900	1.360	8.130	8.320	3,005	599.0	8.90	64	5.9
A-025-168	3.130	1.010	7.870	7.990	3,129	1,342.3	8.90	62	3.9
A-120-024	8.860	2.390	9.580	39.500	3,038	948	9.20	62	4.1
A-120-072	28.700	1.020	33.500	46.000	2,609	3,312	8.90	51	3.8

* EBMUD: East Bay Municipal Utility District

† Cu: copper, Pb: lead, Zn: zinc, DO: dissolved oxygen

‡ MW: makeup water

§ NA: not applicable

Table continued next page.

Table 4 EBMUD water quality data* *(continued)*

Test Number	Parameter[†]								
	Cu *mg/L*	Pb *mg/L*	Zn *mg/L*	Average Current *AC-mA*	Average Resistance *ohm*	mA*hr	pH	Conductivity *mS/cm*	DO *mg/L*
A-120-168	39.000	6.100	44.000	46.500	2,581	7,812	9.80	57	1.5
B-000-168	1.900	0.120	0.500	NA	NA	NA	8.50	64	10.3
B-005-024	0.929	3.020	0.067	0.031	161,290	0.7	8.60	62	9.5
B-005-072	4.380	0.154	0.358	0.030	166,667	2.2	8.30	64	9.7
B-005-168	1.900	0.096	0.050	0.031	161,290	5.2	8.60	63	9.5
B-010-024	1.040	0.457	0.059	0.064	156,250	1.5	8.60	62	10.4
B-010-072	0.456	0.089	0.025	0.062	161,290	4.5	8.50	63	9.6
B-010-168	0.446	0.413	0.022	0.063	158,730	10.6	8.50	66	9.4
B-025-024	1.180	0.559	0.028	0.155	161,290	3.7	8.70	61	10.3
B-025-072	1.070	0.990	0.064	0.155	161,290	11.2	8.60	64	10.2
B-025-168	1.430	0.218	0.074	0.153	163,399	25.7	8.40	65	9.7
B-120-024	2.560	0.049	0.104	0.816	147,059	19.6	8.80	61	9.2
B-120-072	4.450	2.510	0.259	0.793	151,324	57.1	8.50	63	9.5
B-120-168	3.400	0.260	0.600	0.798	150,376	134.1	8.80	63	8.6
C-000-168	1.400	0.570	0.030	NA	NA	NA	8.70	64	10.2
C-005-024	1.880	0.204	0.808	1.000	5,000	24.0	8.70	61	9.2

* EBMUD: East Bay Municipal Utility District

† Cu: copper, Pb: lead, Zn: zinc, DO: dissolved oxygen

‡ MW: makeup water

§ NA: not applicable

Table continued next page.

Table 4 EBMUD water quality data[*] *(continued)*

	Parameter[†]								
Test Number	Cu *mg/L*	Pb *mg/L*	Zn *mg/L*	Average Current *AC-mA*	Average Resistance *ohm*	mA*hr	pH	Conductivity *mS/cm*	DO *mg/L*
C-005-072	3.920	0.560	1.020	0.940	5,319	67.7	8.50	63	9.0
C-005-168	4.000	0.890	3.000	0.830	6,024	139.4	8.70	64	7.7
C-010-024	1.300	0.983	2.110	2.620	3,817	62.9	8.80	64	8.9
C-010-072	1.920	1.190	2.860	2.470	4,049	177.8	8.90	62	8.2
C-010-168	2.720	2.440	5.410	2.650	3,774	445.2	9.00	60	6.1
C-025-024	3.400	0.659	4.260	7.170	3,487	172.1	9.00	62	8.3
C-025-072	2.190	0.940	7.880	6.820	3,666	491.0	9.00	67	6.4
C-025-168	4.310	7.890	9.520	4.910	5,092	824.9	9.30	55	6.1
C-120-024	7.130	3.190	18.000	38.100	3,150	914.4	9.60	63	7.1
C-120-072	7.620	2.480	38.400	34.600	3,468	2,491.2	9.90	67	4.3
C-120-168	11.000	19.000	46.000	42.200	2,844	7,089.6	9.80	55	10.5

[*] EBMUD: East Bay Municipal Utility District

[†] Cu: copper, Pb: lead, Zn: zinc, DO: dissolved oxygen

[‡] MW: makeup water

[§] NA: not applicable

Table 5 PCU water quality data[*]

	Parameter[†]								
Test Number	Cu *mg/L*	Pb *mg/L*	Zn *mg/L*	Average Current *AC-mA*	Average Resistance *ohm*	mA*hr	pH	Conductivity *mS/cm*	DO *mg/L*
5 VAC MW[‡]	0.020	0.002	0.038	NA[§]	NA	NA	7.66	436	0.57
10 VAC MW	0.023	0.001	0.038	NA	NA	NA	7.63	455	1.13
25 VAC MW	0.042	0.002	0.031	NA	NA	NA	7.79	404	0.75
50 VAC MW	0.183	0.001	0.020	NA	NA	NA	7.60	435	0.7
A-000-168	0.909	0.050	4.088	NA	NA	NA	7.518	476	0.82
A-005-024	1.490	0.034	2.070	3.97	1,259	95	7.54	448	1.15
A-005-072	0.367	0.036	2.300	5.43	921	391	7.56	443	0.83
A-005-168	0.403	0.027	2.980	4.98	1,004	837	7.54	447	0.68
A-010-024	0.803	0.040	4.700	19.51	513	468	7.58	519	0.81
A-010-072	0.642	0.064	6.550	16.60	602	1,195	7.51	520	0.62
A-010-168	6.43	0.118	6.85	15.00	667	2,520	7.37	465	0.72
A-025-024	0.891	0.043	5.860	44.20	566	1,061	7.50	517	0.42
A-025-072	1.380	0.076	10.100	32.80	762	2,362	7.48	455	0.65
A-025-168	2.210	0.133	13.300	57.70	433	9,694	7.23	535	0.46
A-050-024	1.140	0.052	4.510	85.80	583	2,059	7.70	579	1.81
A-050-072	0.984	0.088	9.010	93.40	535	6,725	7.43	462	0.8

[*] PCU: Pinellas County Utilities

[†] Cu: copper, Pb: lead, Zn: zinc, DO: dissolved oxygen

[‡] MW: makeup water

[§] NA: not applicable

Table continued next page.

Table 5 PCU water quality data* *(continued)*

	Parameter†								
Test Number	Cu *mg/L*	Pb *mg/L*	Zn *mg/L*	Average Current *AC-mA*	Average Resistance *ohm*	mA*hr	pH	Conductivity *mS/cm*	DO *mg/L*
A-050-168	2.160	0.244	22.900	118.70	421	19,942	7.18	491	0.4
B-000-168	0.938	0.871	4.955	NA	NA	NA	7.548	485	0.75
B-005-024	0.752	0.008	1.130	0.40	12,500	10	7.60	441	1.01
B-005-072	0.576	0.012	1.590	0.40	12,500	29	7.53	429	0.47
B-005-168	0.644	0.014	1.53	0.40	12,500	67	7.66	443	0.7
B-010-024	0.720	0.009	2.330	0.70	14,286	17	7.58	472	1.23
B-010-072	1.370	0.006	4.050	0.80	12,500	58	7.43	528	1.2
B-010-168	1.93	2.55	5.47	0.76	13,158	128	7.44	479	0.75
B-025-024	1.190	0.023	4.540	1.80	13,889	43	7.43	522	0.74
B-025-072	0.967	0.009	3.960	1.80	13,889	130	7.32	475	0.83
B-025-168	0.982	0.037	5.870	1.80	13,889	302	7.27	551	0.94
B-050-024	0.944	0.013	2.820	3.70	13,514	89	7.71	511	1.95
B-050-072	0.239	0.012	2.390	3.40	14,706	245	7.96	444	1.08
B-050-168	0.622	0.023	2.860	3.50	14,286	588	7.63	458	0.59
C-000-168	0.760	2.245	5.078	NA	NA	NA	7.565	486	0.86
C-005-024	0.505	3.370	2.460	7.82	639	188	7.63	431	1.1

* PCU: Pinellas County Utilities

† Cu: copper, Pb: lead, Zn: zinc, DO: dissolved oxygen

‡ MW: makeup water

§ NA: not applicable

Table continued next page.

Table 5 PCU water quality data* *(continued)*

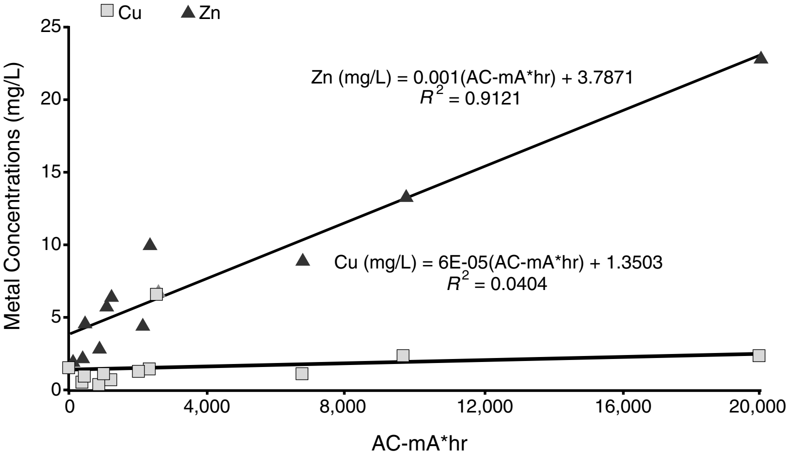
	Parameter†								
Test Number	Cu *mg/L*	Pb *mg/L*	Zn *mg/L*	Average Current *AC-mA*	Average Resistance *ohm*	mA*hr	pH	Conductivity *mS/cm*	DO *mg/L*
C-005-072	0.312	1.990	2.860	5.05	990	364	7.57	449	0.53
C-005-168	0.383	1.200	2.810	4.41	1,134	741	7.63	445	1.12
C-010-024	1.200	1.170	5.700	15.41	649	370	7.59	508	1.16
C-010-072	0.360	1.920	3.420	10.50	952	756	7.65	447	0.5
C-010-168	0.901	0.968	4.4	13.40	746	2,251	7.56	454	0.66
C-025-024	1.630	2.960	13.600	39.10	639	938	7.23	567	0.56
C-025-072	1.850	3.780	11.200	43.70	572	3,146	7.26	484	0.43
C-025-168	0.661	0.758	6.450	41.80	598	7,022	7.53	484	0.7
C-050-024	0.941	1.800	6.220	79.50	629	1,908	7.63	515	1.04
C-050-072	2.710	1.520	22.500	118.50	422	8,532	7.25	377	0.6
C-050-168	0.719	1.600	5.950	80.70	620	13,558	7.66	456	0.94

* PCU: Pinellas County Utilities

† Cu: copper, Pb: lead, Zn: zinc, DO: dissolved oxygen

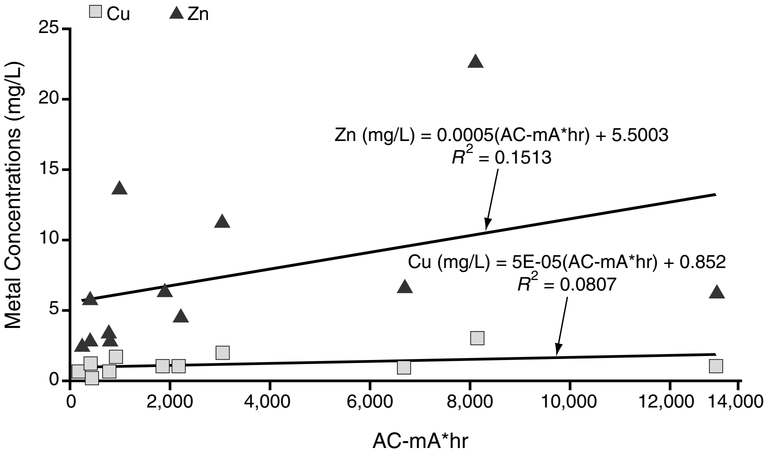
‡ MW: makeup water

§ NA: not applicable



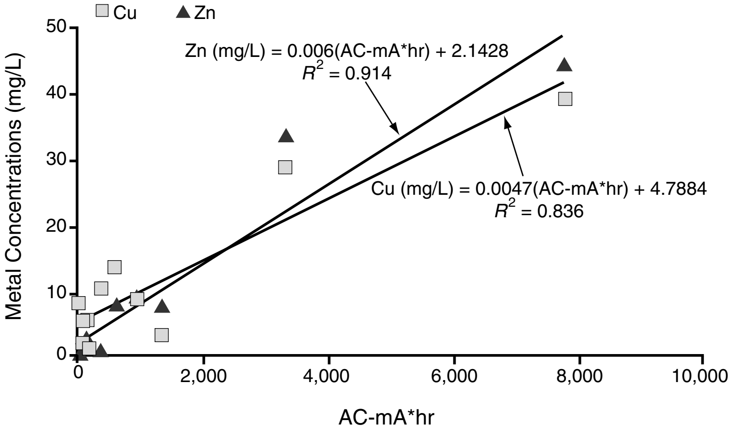
Cu: copper, Zn: zinc, AC-mA*hr: metal pickup of the water as a function of the product of the average mA of alternating current and the contact time, PCU: Pinellas County Utilities

Figure 6 Cu and Zn concentrations as a function of AC-mA*hr for follow-up testing at PCU with type A test articles



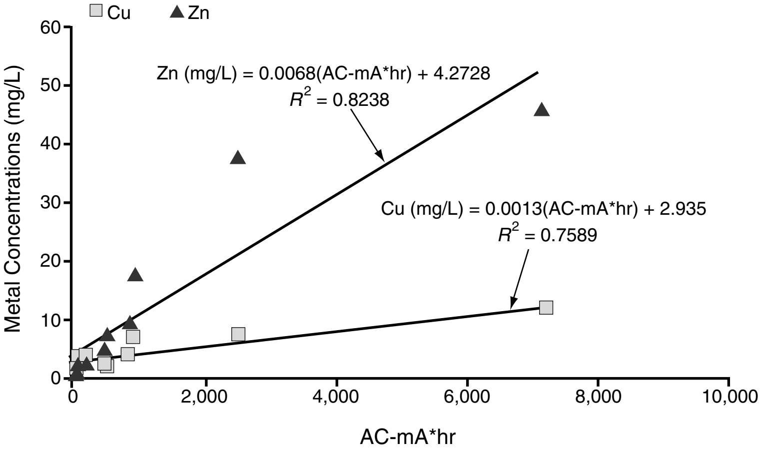
Cu: copper, Zn: zinc, AC-mA*hr: metal pickup of the water as a function of the product of the average mA of alternating current and the contact time, PCU: Pinellas County Utilities

Figure 7 Cu and Zn concentrations as a function of AC-mA*hr for follow-up testing at PCU with type C test articles



Cu: copper, Zn: zinc, AC-mA*hr: metal pickup of the water as a function of the product of the average mA of alternating current and the contact time, EBMUD: East Bay Municipal Utility District

Figure 8 Cu and Zn concentrations as a function of AC-mA*hr for follow-up testing at EBMUD with type A test articles



Cu: copper, Zn: zinc, AC-mA*hr: metal pickup of the water as a function of the product of the average mA of alternating current and the contact time, EBMUD: East Bay Municipal Utility District

Figure 9 Cu and Zn concentrations as a function of AC-mA*hr for follow-up testing at EBMUD with type C test articles lower metal concentrations were directly attributable to the higher resistance and thus lower currents present in the test systems. EBMUD showed significant Cu pickup with very little Zn increase.

Pb release in the EBMUD B series was sporadic and in some cases significant. The source of the Pb in the EBMUD tests was probably brass plugs in the ends of the test articles. PCU data for B series test articles showed low Cu pickup but significant Zn release. The source of Zn in the system was not clear and may be related to solder fluxes used during fabrication. Pb levels were low for the PCU B series tests, which used plastic plugs rather than brass plugs to seal the end of the test article.

Description of C Series Test Articles

The C series test articles consisted of copper tubing separated by a dielectric union with 50:50 lead-to-tin solder surfaces exposed to the water. For this series, Cu, Zn, and Pb increased with increasing AC-mA*hr for both the EBMUD and PCU data. The level of Cu release was lower for PCU than for EBMUD, despite the fact that higher currents were present at PCU. Levels of Zn were somewhat higher for EBMUD, but it should be remembered that EBMUD test articles were exposed at 120 V, whereas PCU was limited to 50 V as the highest voltage level. Pb increases for PCU were consistently higher, compared with Pb increases for EBMUD.

DISCUSSION

The results of the follow-up tests at EBMUD confirmed the results of initial testing. Cu, Pb, and Zn release was significant and in many cases exceeded the ALs for primary or secondary drinking water standards. Differences in metal release behavior between the PCU and EBMUD tests are obvious and indicate that the differences were probably attributable to the different sources and treatment of water at the two utilities.

Other researchers have postulated that the mechanism of AC corrosion is caused by rectification of AC to DC by “metallic” rectifiers at the metal oxide interface (Bruckner, 1963; Waters, 1962; Kulman, 1961). In metallic rectifiers such as Cu–copper oxide, the rectifying junction is between the oxide semiconductor and the metal surface. The direction of the current is from the semiconductor to the metal. For Cu surfaces, copper oxides act as the semiconductor on the Cu metal surface and produce a half-wave rectifying circuit, which results in net DC.

PCU uses groundwater as its source, whereas EBMUD draws on surface water. PCU water used in the testing had much less dissolved oxygen (DO = ~1 mg/L) compared with EBMUD water (DO = ~10 mg/L). Because the formation of copper oxide is related to the amount of oxygen present in the makeup water and DO would be consumed in corrosion reactions that are causing metal release, the reduction in DO was correlated with the increase in metal ion equivalents in the solution. For the EBMUD conditions of higher initial DO, good correlation was found between increased metal ion content of the water and reduction in DO. The correlation does not hold for the lower initial DO PCU data. This suggests that DO plays a role in the magnitude of the release of metal.

In addition to the differences in DO, PCU uses a blended phosphate corrosion inhibitor to control Cu corrosion. Although EBMUD does not use a corrosion inhibitor, the utility does use lime and sodium hydroxide to increase alkalinity and pH to around 8.5 to control Pb and Cu corrosion. Both utilities' water treatment programs are clearly aimed at reducing Pb and Cu. The presence of the corrosion inhibitor at PCU probably contributed to the differences in measured metal release, because of differences in sequestering characteristics of the source and treated waters.

Visual Examination, Microscopy, and Microanalysis of Selected Test Articles

At the completion of the test program, the test articles were sectioned longitudinally and visually examined. Figure 10 shows the interior surface of a typical test article with a dielectric union after 168 hr of exposure to 120 VAC in EBMUD water. Evidence of the effect of applied voltage can be seen in the vicinity of the narrow gap in the metal. Selected test articles were sectioned after testing and examined using scanning electron microscopy (SEM). Areas on each sample examined by SEM were chemically analyzed using qualitative energy-dispersive X-ray analysis (EDXA).

The top left SEM photomicrograph in Figure 11 shows the Cu side of the dielectric union from the EBMUD A series test articles exposed to 120 VAC for 168 hr. The top right SEM photomicrograph shows the Cu side of the dielectric union from the PCU A series test articles exposed to 50 VAC for 168 hr. The surfaces of the Cu are similar in appearance for both the EBMUD and PCU samples. However, EDXA analysis of the surfaces indicated a significant amount of calcium present on the PCU Cu side compared with the PCU brass or EBMUD brass and Cu surfaces.

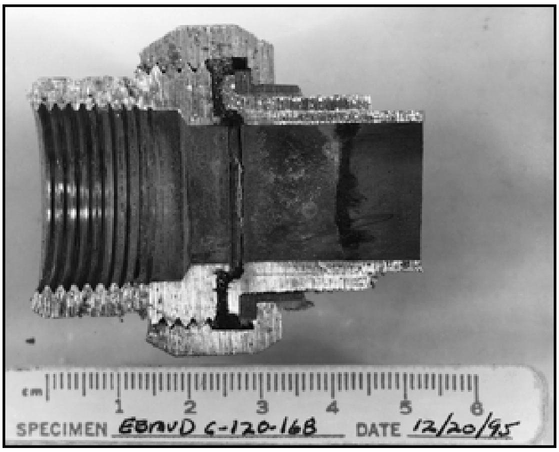


Figure 10 This photograph shows brass and copper sides of a dielectric union for the East Bay Municipal Utility District A series exposed to 120 V of alternating current for 168 hr.

The bottom left and bottom right SEM photomicrographs in Figure 11 show the Cu surfaces adjacent to the PVC section for the B series tests at EBMUD and PCU, respectively. The test articles from the EBMUD B series were exposed to 120 VAC for 168 hr. The test articles from the PCU B series were exposed to 50 VAC for 168 hr. The surface of the EBMUD B series sample was relatively clean, smooth, and featureless. The surface of the PCU B series test articles had a green deposit. EDXA analysis of the surfaces shown indicated significant amounts of chlorine (most likely as chloride), Zn, and tin on the PCU Cu surface. The EBMUD Cu surfaces for the B series of tests showed no indication of Zn or tin and only a very slight chloride (most likely as chloride) signal. It is most likely that this deposit was attributable to residual solder and solder flux used to fabricate the specimens. A slight indication of calcium was observed on the PCU B series Cu sample.

Direct comparison of the PCU and EBMUD C series samples was not possible because of differences in the utilities' fabrication methods for the test articles. It appears, however, that the higher chemical content and hardness of the PCU water increased the precipitation of calcareous compounds (scales), which may account for the inhibited internal corrosion. The effect of the blended phosphate corrosion inhibitor used at PCU could not be determined from these tests. Data seemed to indicate that EBMUD low-hardness water with little or no buffering

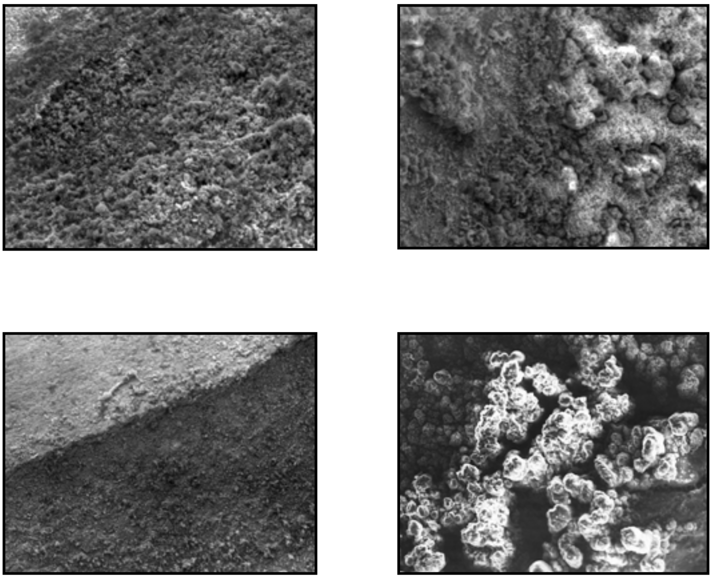


Figure 11 These SEM photomicrographs show the copper side of dielectric union for the East Bay Municipal Utility District (EBMUD) A series exposed to 120 V of alternating current (VAC) for 168 hr (top left), for the Pinellas County Utilities (PCU) A series exposed to 50 VAC for 168 hr (top right), the copper surface adjacent to polyvinyl chloride (PVC) for EBMUD B series exposed to 120 VAC for 168 hr (bottom left), and for PCU B series exposed to 50 VAC for 168 hr (bottom right).

capacity resulted in higher metal release, despite higher resistivity and therefore less resulting current.

SUMMARY AND CONCLUSIONS

The past few decades have seen an increase in the use of plastic service piping and other dielectric or electrically insulating connections in water services. Electrical resistance or insulation in the normally conductive pipe will force some current to flow through the parallel internal water and external soil paths. Discharge of current on the inside of the pipe increases metal concentrations in the service line.

Two series of tests were conducted to investigate the effect of electrically insulating unions on water quality in the presence of applied

AC voltages of 123 and ~50 V. Data from these tests yielded the following conclusions.

- AC voltage had a pronounced effect on Cu release for both the 123- and 50-VAC tests. Pb and Cu contents measured after 24 hr of exposure at the lower applied voltage exceeded USEPA ALs of 0.015 and 1.3 mg/L for Pb and Cu, respectively.
- Electrical insulators can affect water quality in terms of metal release. Limited tests were run with copper tubes having brass dielectric unions using applied AC voltage levels from 5 to 123 VAC; in some cases, after only 24 hr of exposure, this produced Cu concentrations in potable water exceeding USEPA ALs. In actual practice, resistive, but not insulated, connections may force the current to leave the pipe and increase metal concentrations at the customer's tap.
- Metal release generally increased with charge transfer, in accordance with Faraday's law.
- On the basis of the metal release data collected in these limited tests, the AC corrosion rate was calculated as 0.14% of the corresponding DC rate for the conditions investigated here.

Recommendations

On the basis of these findings, the authors developed the following recommendations.

- When a water supplier can detect stray AC and/or DC on water mains and services, the power provider should be notified immediately of a potential hazard and be requested to inspect and if necessary repair suspect neutrals.
- When a water supplier exceeds the Pb and Cu AL concentration in first-draw water samples at the consumer's tap, the supplier should examine and evaluate the electrical and piping system for high levels of AC and/or DC to determine whether the electrical system is contributing to the metal release, particularly if the utility practices a treatment technique for internal corrosion control as required by the LCR.

ACKNOWLEDGMENTS

The authors are indebted to the water treatment utilities that cooperated and participated in this project, which was funded by the AWWA Research Foundation (AwwaRF) under contract 913-93. The technical assistance and persistence of Dana Drath and Ron Bianchetti of East Bay Municipal Utility District were invaluable to the design and performance of the tests; other significant contributions were made by Robert Powell and the staff of Pinellas County Utilities. In addition, the authors thank AwwaRF project officer Roy Martinez and the Project Advisory Committee for their help and advice.

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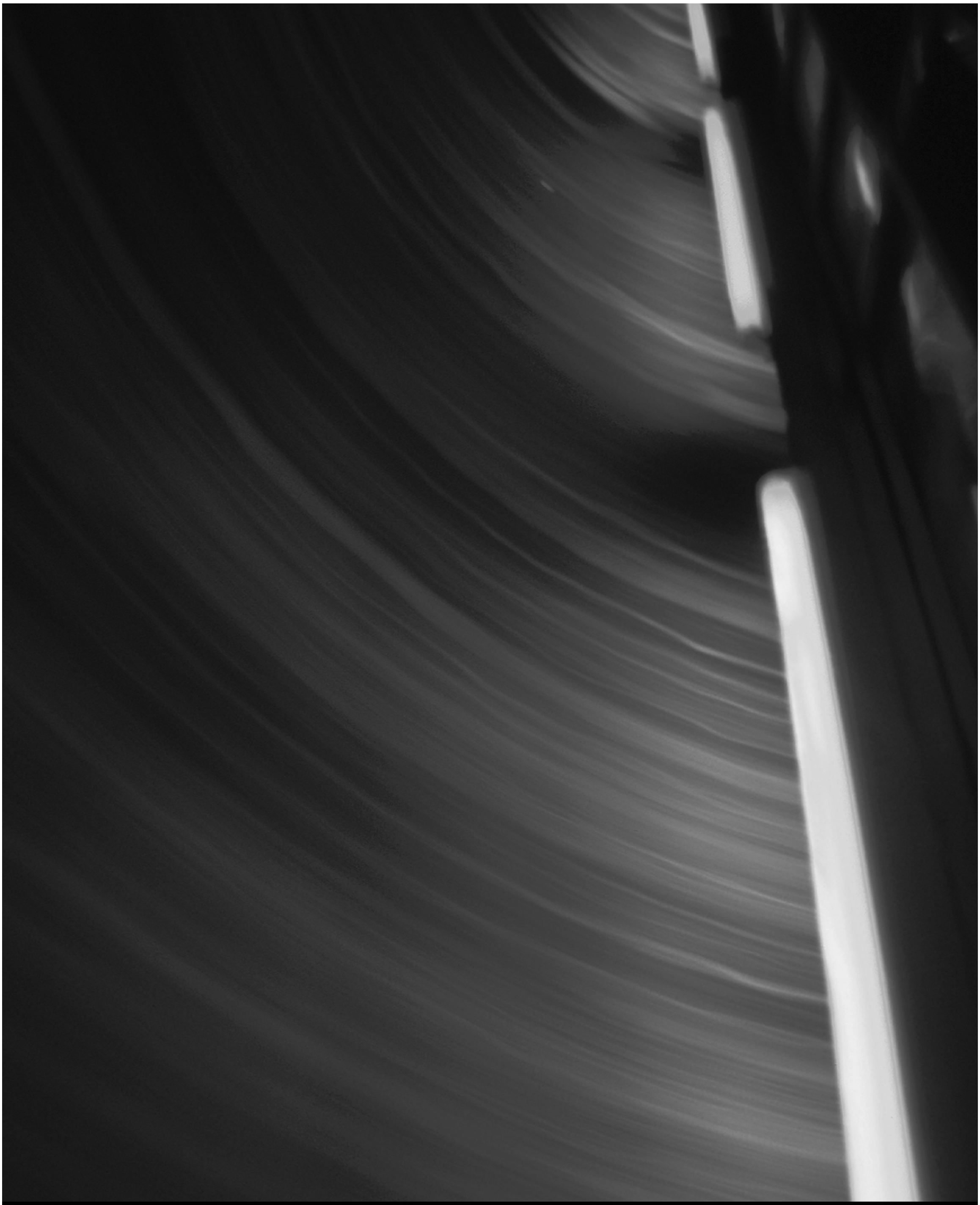

Part 6

Rapid or Real-Time Monitoring

CHAPTER · 25

A Vision for Real-Time Monitoring and Modeling of Water Quality in Water Distribution Systems

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INTRODUCTION

Several business and safety imperatives have led drinking water distribution companies to consider deploying real-time monitoring and modeling systems:

- The public's expectations for a plentiful supply of wholesome drinking water available "on demand" from the tap continue to increase.
- The public expects fast and accurate information about conditions in parts of the network that impact them directly.
- Vulnerability assessments and other studies reveal the need for early warning and emergency response systems. The intent is to provide better preparation for system contamination or other potentially disastrous incidents.
- Drinking water regulations require more regular and continuous sampling and monitoring to ensure compliance with quality requirements.
- Energy prices also provide incentive to provide better monitoring and response capabilities to reduce operating costs.

This chapter presents a vision for real-time monitoring and modeling of water distribution system hydraulics and water quality. A distinct aspect of this vision is its incremental nature. Organizations can begin to benefit from the first steps of simple monitoring and modeling, and choose to implement further pieces over time.

This chapter also addresses some of the key questions of organizations considering the adoption of real-time or near-real-time monitoring and modeling practices:

- What exactly do we mean by "real-time monitoring and modeling"?
- What are the benefits and value associated with such a system?
- What questions does it allow us to address?
- What operations are necessary to use the system?
- Natural gas and petroleum pipelines have been conducting real-time modeling for years. What lessons can we learn from the pipeline industry experiences?
- What is the effort involved?
- Who in the organization uses it?
- Who manages it?

BACKGROUND

A real-time system is a computerized system that runs alongside ("in real time" with) a real-world process (man-made or natural). The system has some kind of time constraints upon its performance that differ based upon the needs of the situation. Data and/or directives are transmitted between the process and the real-time system. These systems must be "fast," generally striving to provide up-to-date information, to remain synchronized with a real-world process, and perhaps to control it.

Monitors "watch" something, and *real-time monitoring systems* provide timely tracking of various key indicators. Control systems such as SCADA (Supervisory Control And Data Acquisition) combine monitoring with the ability to interact with the process being monitored.

Real-time modeling systems simulate the true process generally using computer technology. Generally, real-time monitors must feed these modeling systems. A real-time modeling system presupposes that real-time monitored data are available to support the predictive simulation.

Real-time systems are used for a variety of purposes, including security, process control, and as embedded systems. For example, security systems monitor various conditions and provide feedback whenever conditions change—the concepts of video surveillance and alarms apply here. Air and space industries use real-time and embedded real-time systems to aid with flight, responding immediately to rapidly changing conditions. Stock traders use real-time systems to keep up with trading prices and to time their actions in the market. Real-time systems are also used in a variety of other industries for things like manufacturing process control, pipeline transmission control, electrical transmission, and distribution control, etc.

To date, the primary uses of real-time systems in the potable water industry (limiting the scope to just treatment and distribution) include treatment plant operation control and equipment control. SCADA systems are widespread, offering effective monitoring and control for water distribution, pump run optimization, treatment plant operation, and ordering water at different sources.

SCADA systems collect massive amounts of data, representing potential add-on benefits to the water company if those data are exploited. In addition to the primary goal of reliable delivery of drinking water, historical and real-time operational data could be used as follows:

- Coupled with hydraulic modeling and optimization of pump schedules, use the data to improve efficiency and cost-effectiveness of operations.
- Hydraulic and water quality modeling with SCADA data may allow progressive operational changes to improve delivered quality of water.
- For systems with significant wholesale aftermarkets, SCADA data plus modeling and forecasts/water orders can help ensure timely delivery of water to wholesale purchase points.

Real-time modeling systems have seen widespread use in petroleum, natural gas, and electric utilities. To date, they have been theorized for water systems, and some have been deployed by Advantica (then named Stoner Associates) and others.

In 1993, Dupage Water Board in Illinois deployed a system to help with delivery to wholesale water purchasers [1]. In 1995, a more expansive system was deployed at the Générale des Eaux–operated concession in Paris, France, with the added goal of monitoring and improving delivered water quality [2]. In 2001, a similar "on-line" (real-time) modeling system was deployed for the City of San Diego Water Department.

In 1998, the American Water Works Research Foundation, the Electric Power Research Institute–Community Environmental Center (EPRI-CEC), and East Bay Municipal Utility District (EB-MUD) collaborated to produce a requirements model for an energy and water quality management system [3]. This research advanced to a standard set of specifications to build and provide components for the system [4].

SAGEP, the authority responsible for overall management of the Paris, France, water network, makes use of "virtual sensors"—monitors displaying calculated values along with the real measurements—in its control centers [5]. SAGEP has also deployed continuous chlorine monitors in roughly 100 key locations throughout the Paris network [5].

With these and other initiatives, application of real-time monitoring and modeling systems for water systems appears to be increasing. The business needs that are driving this increasing interest in the water industry include the following:

- Rising energy costs demand more efficient operations.
- With increasing complexity of operations, tools are available to help with understanding and managing the system.
- Fewer experienced operations personnel means that tools are needed to train and make operators more effective.
- More stringent regulations on quality and delivery increase the need for careful, well-planned operations.
- Security concerns are causing water purveyors to examine ways to improve monitoring and early warning systems.

A REAL-TIME, ON-LINE SYSTEM FOR WATER DISTRIBUTION

Water distribution system companies have a mission to deliver drinking water:

- Of safe quality
- Within regulatory constraints
- In sufficient quantity and at sufficient pressures to meet customer demands
- Economically
- Securely

These conveyance systems are often large, complex, and expensive to operate and maintain. Many functional groups in the water company need the ability to monitor and assess performance, including long-range planning, water resource, operations, maintenance, and administration.

Most water systems of significant size collect massive amounts of operational data through SCADA and other means. However, these data are rarely available in a timely fashion to anyone but the dispatch operators on duty. Making status and forecast conditions available more freely and timely within an organization will be of great value.

Working with the city of San Diego, Advantica (Carlisle, Pa., and Loughborough, UK) have developed an on-line, real-time system called the System Status Advisor (SSA). Part of this system is in place, and the remainder will be deployed in 2003–2004. Figure 1 shows the main components of the initial system.

The entire SSA runs on a cycle driven from the SCADA system. A small custom application periodically gathers information from the SCADA system and feeds these current data into the system via a handoff file.

A hydraulic simulation runs continuously from within the system. The SynerGEE® OnLine Module (OLM) overlays the SCADA data on the modeled system, adjusting pump states, tank and reservoir levels, and demands, and then the analysis is run. Each analysis produces an echo of sparse field data, plus system-wide detailed hydraulic simulation results for use by authorized individuals within the utility through the corporate intranet site. In this way, the hydraulic model provides solid estimates of the entire system status, "filling in the blanks" between the system monitoring points. Note that, at San Diego, the entire service area (over 3,500 miles of water lines) is modeled in detail, providing an excellent level of detail on the system.

The core of the system is an Oracle® database serving as an information "warehouse." This main repository of information collects SCADA data and modeling results, storing them in time-stamped records and making all the data available to data consumers in the organization. This enables trending, queries of historical information, and some kinds of "data mining."

The Web server assembles data from the database into Web-consumable tables, charts, and other forms. It also uses the corporate Geographic Information System (GIS) to prepare and present read-only map views of the system. The Web views provide alarms that are optionally configurable by the end user. These custom alarms will provide earlier warning of undesirable conditions in the distribution

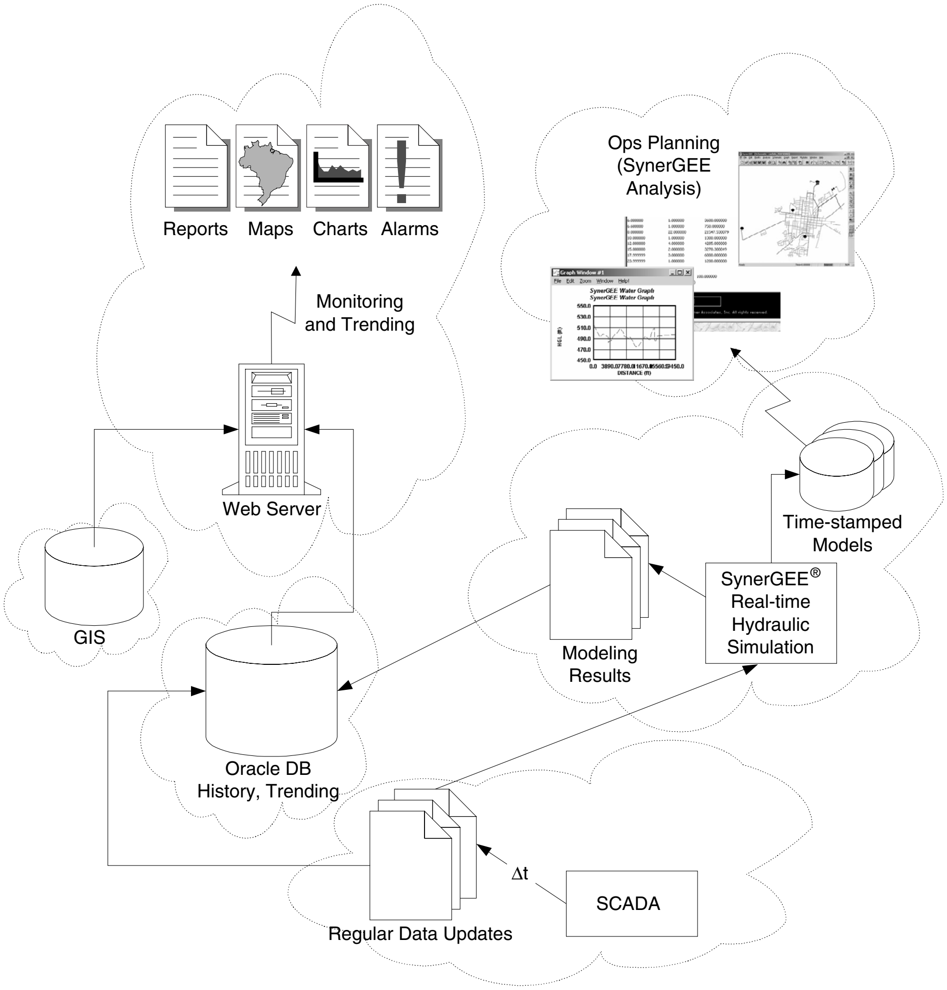
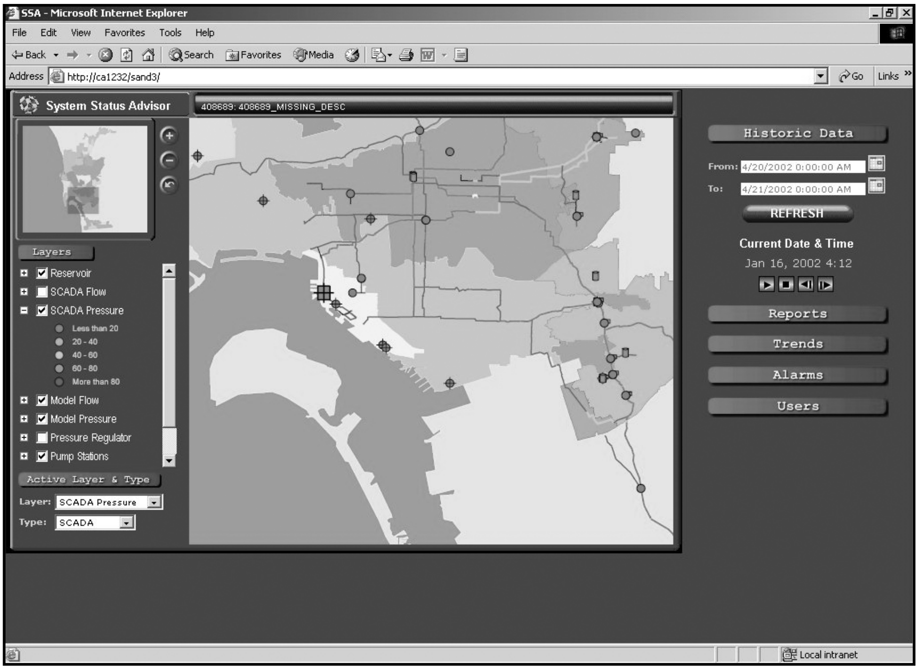


Figure 1 Diagram of the System Status Advisor

network. These alarms can be based upon the SCADA data, simulation results, or some combination, resulting in richer feedback than provided by the SCADA system alone. As users better learn their system's behavior, better alarms can be devised and nuisance alarms discarded.

Standard and custom displays are available from the Web server (see Figure 2, for example). Users in the organization can use their Web browser to find just the information of interest, updated periodically. These displays can be tailored to the needs of the user.



Address: http://ca1232/sand3/

Layer: SCADA Pressure

Type: SCADA

From: 4/20/2002 0:00:00 AM

To: 4/21/2002 0:00:00 AM

Current Date & Time: Jan 16, 2002 4:12

Figure 2 System Status Advisor main interface

The system functions in a cycle with a frequency that can be tailored to the situation:

- Current SCADA data are made available to the system.
- The simulator overlays the SCADA data onto the model, adjusting various operational data to match the SCADA input.
- The simulator produces current hydraulic results for the entire modeled area (at San Diego, the entire service area).
- The Oracle "warehouse" pulls the SCADA and simulation values into the database, checks alarms, and informs the Web server of updated data.
- The Web server updates its dynamic views based upon new information.
- All components wait for more SCADA data or user input.

This real-time, on-line system is modular and readily expandable. New views for different purposes can be incorporated easily. The central "data warehouse" can consume data from other data sources, corporate or otherwise (for example, USGS stream gauge data). New processes can be added later; in fact, the system is planned to incorporate water quality modeling, Customer Information Service (CIS), and laboratory information management systems (LIMS) system data in 2003–2004. By incrementally expanding, this system also could eventually incorporate water demand forecasting, helpful security alarms, energy optimization, etc. For example, Figure 3 shows how water quality modeling and demand forecasting data would plug in to the system.

To enable water quality, the SCADA data collected are enhanced to include source quality measures (chlorine residual, etc.). The simulator and model are enabled for water quality tracing. Water quality data are added to the results storage directives into the historical database and displays.

To enable forecasting, a Forecaster is plugged in as a component. Its data are overlaid on the simulator and also pulled into the historical data DB. The SSA will enable water companies to collect and analyze a large volume of data to detect trends. These data may be precise or merely indicative. In either case, this system is intended to help operators, engineers, and managers learn, manage, and protect their systems more effectively, allowing the investigation and resolution of issues such as:

- Poor pressure and other customer complaints
- Storage and pump cycling operational issues
- Operational costs due to high power consumption
- Identifying and understanding various, previously unnoticed trends

As the vision expands and new components are deployed and enabled, other questions can be dealt with more effectively, including:

- Excessive residence time of water in storage and in regions of the system
- Low or highly variable chlorine residual
- Looking forward using forecasted demands to prevent future delivery shortfalls or other operational events
- Quality (appearance, taste, or odor) complaints

With the deployment of real-time sampler monitors out in the distribution system, early warning and tracking of contamination incidents and improved response might also be possible.

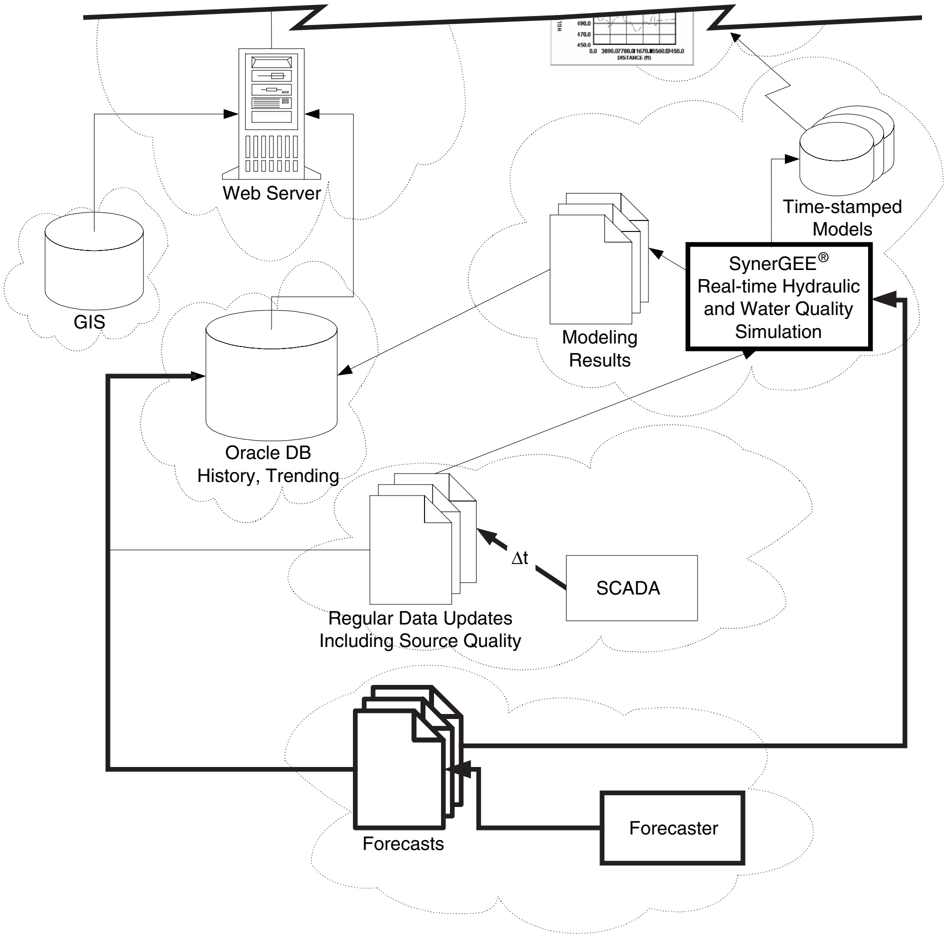


Figure 3 Expanding the system with water quality modeling and demand forecasting

Who are the users of this system? The answer to this question depends upon the desires and strategy of the company deploying it, of course. However, it can be surmised that:

- Planning and operations engineers will use the system to better understand normal operations and to try new operational strategies before recommending or applying them.

- Operators will also use it to be more informed of the system status.
- Water quality professionals will learn of, or gain insight into, local and temporal water quality issues.
- Managers will have custom screens giving, in summary form, recent system performance and current status, with more information available through browsing.

REAL-TIME WATER QUALITY SIMULATION

It is increasingly important to understand the temporal and spatial patterns of water quality in drinking water distribution.

In water distribution, most SCADA and telemetry systems monitor hydraulic and operational data such as pressures, flows, tank levels, and pump modes. Rarely are water distribution systems instrumented to monitor water quality parameters such as residual chlorine, TOC, pH, turbidity, etc. Such measurements generally are made at the treatment works. It is helpful to have a sense of the relative quality of water out in the water distribution network, but very few companies currently have the means available to know this in real time.

Water quality tracing simulations have been available for roughly 15 years now [6]. In the United States, they were initially used experimentally or to evaluate contamination incidents. These uses remain, but today, companies are building water quality models to help with regulatory compliance, to understand relative system security from contamination, and to provide higher customer safety and satisfaction.

Water distribution systems are dynamic "reactors." Quality conditions change regularly. Quality professionals strive to provide water of consistent quality to all customers. Consistency increases satisfaction. Consumers notice fluctuations in chlorine residual, taste, odor, and color; these changes influence their satisfaction and confidence in the water supply. While change in residual drives customer satisfaction with the utility's product, maintenance of proper residual and disinfection by-product levels is the utility's core charter.

It may be possible to use water quality models coupled with real-time SCADA data to predict (at least in an indicative sense) the temporal and spatial quality of water, and changes in it, in the distribution system. Working with the City of San Diego water model, Advantica has prototyped a way to create this kind of information. The data and system requirements are:

- A calibrated hydraulic distribution model
- A reaction model (if chlorine or other reactive substance is to be tracked; source and pipe wall *reactivities*—coefficients describing the rate of substance consumption or growth—must be provided)
- SCADA data:
  - Quality measurements at the treatment plant discharge or wherever water enters the distribution system (these will be used to provide source quality information to the water quality model)
  - Hydraulic measurements: flows, tank levels, pump speeds, etc., sufficient to allow accurate real-time modeling

The steps in the analysis are as follows:

1. The SCADA data are overlaid onto the hydraulic model at a regular time interval.
2. The hydraulic measurements are used to produce a set of "connected steady-state" hydraulic results.
3. The quality measurements are used to update the source quality inputs to the model. This, coupled with the hydraulic results, is used to compute a dynamic water quality time step, updating concentrations throughout the modeled system.

Note that the water quality results are connected over time, with concentrations moved about by Lagrangian formulation. The hydraulic results are fundamentally steady-state, but the date and time information is maintained to ensure proper water quality computations and to enable time trending in graphs and reports. While other analysis approaches are possible, this method has the advantages of producing a close representation of current hydraulics at any given "snapshot" in time, while maintaining the time-dependent "memory" in the water quality analysis.

APPLYING THE VISION IN SAN DIEGO

The City of San Diego Water Department (CSDWD) serves more than 1.4 million people populating more than 331 square miles of developed land. San Diego's water infrastructure represents one of the most complex water delivery systems in the United States.

In addition to its three water treatment plants, San Diego maintains and operates more than 3,500 miles of water lines, 45 water pump plants, 100-plus pressure zones, and more than 200 million gallons of potable water storage capacity in 28 standpipes, elevated tanks, and concrete and steel reservoirs.

San Diego is located in the semiarid desert region of the southwestern United States, where rainfall can vary to extremes. On average, San Diego must import nearly 90% of its water from other areas, specifically northern California and the Colorado River.

The detailed hydraulic water model is being used daily to support long-range planning and short-term operational needs. The Planning and Project Development Section of Water Policy, CIP Finance, and Planning Division of the CSDWD consists of 15 full-time staff members each working to confirm the CSDWD's commitment to a safe and reliable water system at the lowest capital investment.

The CSDWD's overall vision was to have an integrated system that links water modeling, the GIS, CIS, SCADA, and LIMS so that system planning, operations, water quality professional, and managers can receive the maximum benefit from all of these computer information systems.

The creation of the detailed water model started in October 1997 and was completed by April 1999. The CSDWD's mapping system is maintained on a GE Network Solutions (formerly SmallWorld) GIS. As intended, this project relied heavily on data from GIS. An important part of the detailed water model was the placement of each customer's demand onto the piping network or the link to the customer information system. In August 2000 the detailed model was calibrated for a 24-hour maximum day demand and the model was adjusted to match the field condition.

In May 2001, CSDWD integrated the detailed water model to SCADA using Advantica's OnLine Module (OLM) of the SynerGEE® water hydraulic analysis desktop. The deployment of the OLM has helped the planning and operation staff with faster recognition/response to system emergency situations, has improved timeliness and confidence in modeling data and calibration, and has improved the system operation through better information and reporting. In 2003, further benefits are anticipated with the development and application of a calibrated water quality model using SCADA data.

CSDWD engineers maintain the hydraulic model, updating it manually as the field conditions change (weekly or monthly). Upon each update, the new model becomes the new basis for OLM simulation.

SynerGEE OLM runs on a standalone PC, and planners and operators have remote access to its models and other results. A "current conditions" model and results are available at all times. This will help the operators with the system operation unknowns. The on-line model is also used by planning staff for performing hydraulic analysis on major water facilities upgrades, shutdown, system operation planning, system operation changes, source of water contribution, and system deficiencies.

A SCENARIO FOR USE OF REAL-TIME WATER QUALITY

To demonstrate the value of real-time monitoring and modeling of water quality, a usage scenario for the San Diego system is presented.

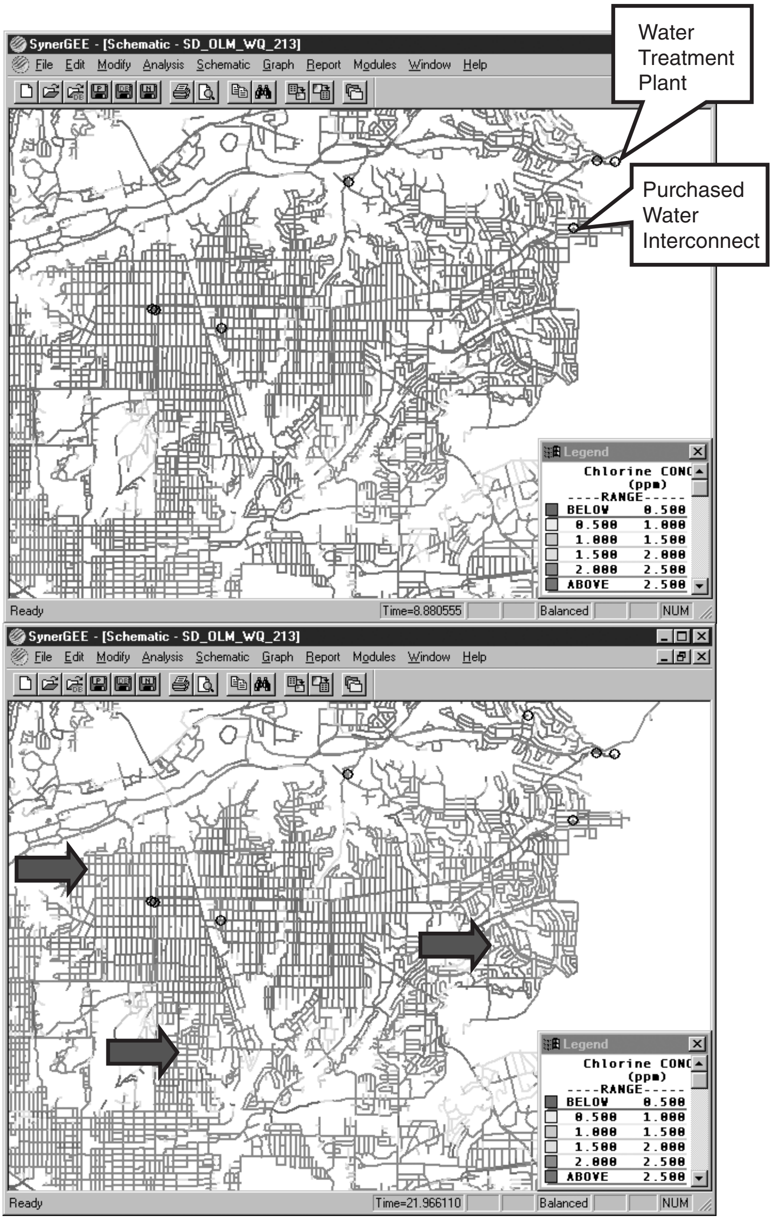
The CSDWD SCADA system monitors tank levels, pump speeds, control valve positions, and metered supplies. Also, the chlorine residual in the water is measured at the plant discharge points; in this scenario, it is assumed that these measurements are added to the SCADA system. Also, purchase-point connections are assumed to have chlorine residual on SCADA so that all points of water intake are monitored. No other monitoring of chlorine residual is done out in distribution, so the real-time system is used to predict chlorine residual and age of water based upon hydraulics and estimated/calibrated reactivity coefficients. Finally, it is assumed that regular jar tests are done at all supply points to determine the rate of chlorine consumption (the reactivity coefficients). From this, reactivity coefficients for the various supplied waters can be regularly applied to the real-time model.

Suppose the operators, examining the displays from the monitoring and modeling system, notice diminishing chlorine residual in certain regions throughout one day. Figure 4 illustrates this; compare the map themes from 9 a.m. and 10 p.m. colored by chlorine concentration.

Exploring further, the operator might check some time trends at various points in the model. Figure 5 shows modeled chlorine concentration at three sites. The dip in chlorine residual is appearing shortly after noon in some sites, into the evening in others.

Personnel might be field-deployed to take samples and determine if a problem really exists. If the tests reveal no problem, this information could be noted; more rigorous field tests might be scheduled to gather data and retune the model to produce more accurate predictions. However, suppose field tests confirm the diminished chlorine residual noted in the model.

Recognizing a problem has developed and is spreading, operators note that, at just before noon, one of the water treatment plants had



Chlorine CONC (ppm)

----RANGE-----		
BELOW		0.500
0.500		1.000
1.000		1.500
1.500		2.000
2.000		2.500
ABOVE		2.500

Chlorine CONC (ppm)

----RANGE-----		
BELOW		0.500
0.500		1.000
1.000		1.500
1.500		2.000
2.000		2.500
ABOVE		2.500

Figure 4 Chlorine concentration at 9 a.m. and 10 p.m.

curtailed its output for maintenance, and a purchased water interconnection had increased its flow to provide the necessary supply. Figure 4

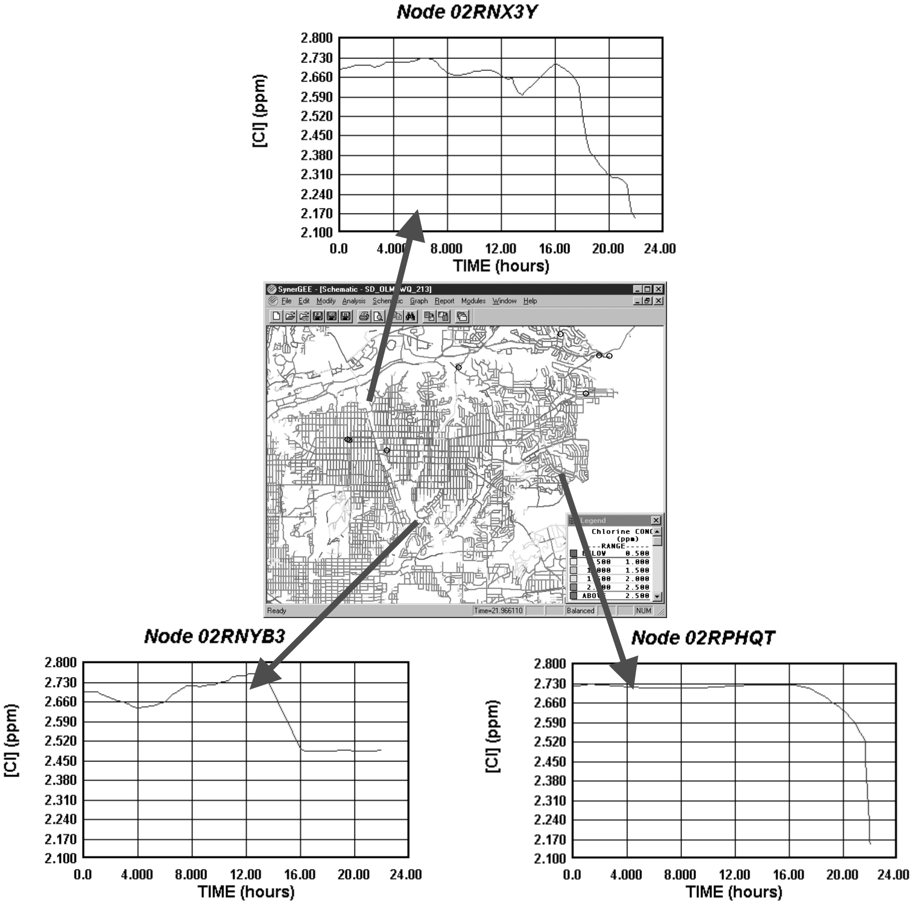


Figure 5 Chlorine concentration over time

shows the two sources; the North source is the curtailed supply from the water treatment plant. Since this operational change might be the cause of the drop in residuals, the lab might be called to confirm the incoming chlorine concentration figures. Finding that the water is supplied at 3 ppm (the expected, accepted level), the source's chlorine demand characteristics are explored. Normal jar test data are consulted, and it is noted that this source's first-order reactivity with chlorine is 0.05/hr—higher than that coming out of the nearby water treatment plant. The model is consulted to see if this is the cause of the dip in chlorine residual. Figure 6 dramatically confirms that the increased flow from the

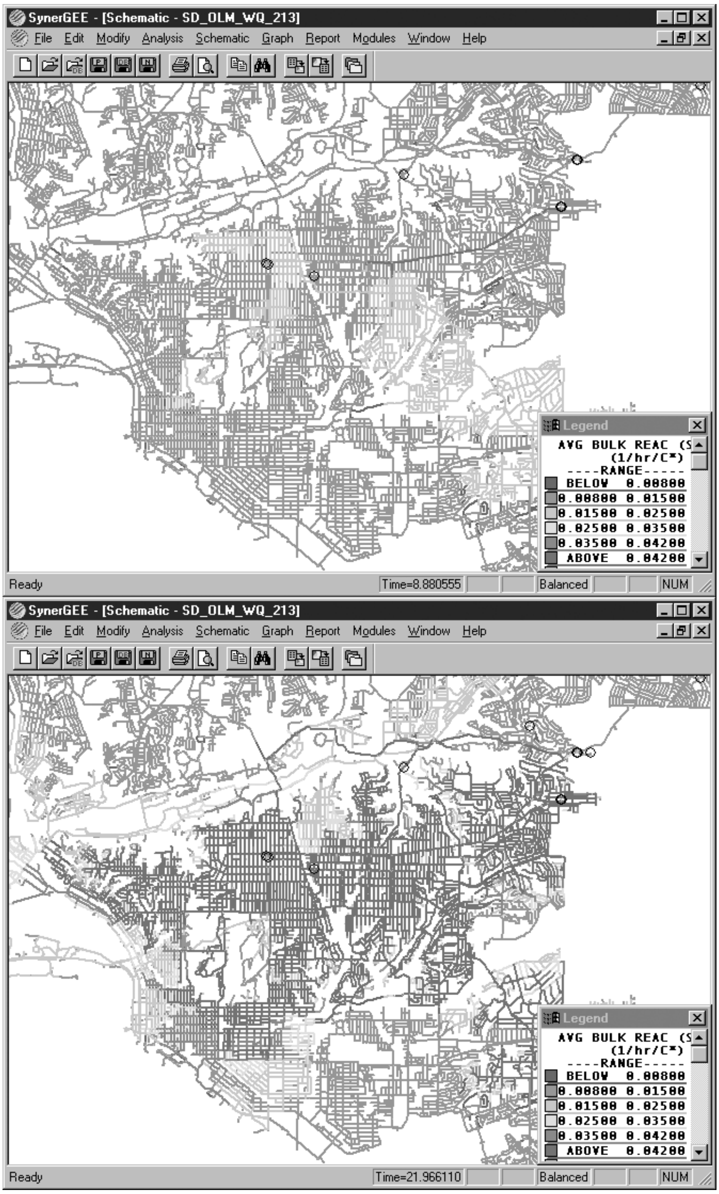


Figure 6 Bulk reactivity in water, 9 a.m. and 10 p.m.

purchased water source has substantially increased the chlorine demand in the region. As the purchased water replaced the treated water, more chlorine was being consumed.

Having determined the cause, options for correcting the situation could then be explored. Options for addressing this situation might include:

- Meeting the temporary demand needs through a different supply, if available
- If chlorine treatment is already done at the interconnect, increasing the dosage to offset the increased consumption (long-term; install chlorine booster if not presently available)

Taking the "current conditions" model into SynerGEE®, an engineer could explore each of these options through modeling scenarios to see which operational option best rectifies the situation. Once a satisfactory approach is selected, this pretested operational change could then be applied.

Over time, the water company might choose to deploy real-time sampling stations out in the distribution system. The model could be used to help locate these stations effectively to provide good coverage of the distribution system. Once the samplers are operational, the continuous comparative data could allow regular tuning of the water quality model, perhaps using automated techniques. Further, these sensors, coupled with the model, might together provide an effective early warning system for system quality upsets.

Whether predictive or indicative, computing water quality with real-time data can enable proactive management of water quality, with the potential of decreasing the risk of customer illness and dissatisfaction.

REAL WORLD ISSUES WITH REAL-TIME MODELING SYSTEMS

Real-time modeling systems promise tremendous support in the effective operation of water distribution systems. They do, however, offer particular challenges when deploying and maintaining them. Advantica has deployed a variety of real-time and near-real-time modeling systems in the water, natural gas, and petroleum industries. Following is a summary of the primary deployment and maintenance issues with such systems.

The term "real-time" indicates that some time interval exists (generally seconds or minutes) between consecutive SCADA scans. However, it is not uncommon for temporary data or communications outages to occur, rendering SCADA monitoring information incomplete. Even if the data are complete, they contain uncertainties due to sensor error, etc.

Moreover, it is common in water systems for the SCADA data to be incomplete: that is, the modeling system is not given complete data to enable it to deduce the entire system state. SCADA systems often have a wide variation in "scan rate," resulting in data collected at a variety of periods, and no one time (or very few times) at which every measurement is known. Some key measurement may not be telemetered—for example, a tank level. Also, very few flows typically are telemetered in water distribution systems. All systems have some amount of unaccounted-for water as well. This combination of issues makes accuracy difficult (though not impossible) to achieve with real-time modeling.

The biggest challenges, then, are for real-time modeling systems to deal with data incompleteness and data uncertainty. There are a variety of approaches to dealing with these issues, including data scrubbing and state estimation techniques. It is beyond the scope of this chapter to address these techniques. Advantica has applied a variety of approaches to different real-time systems, depending upon the needs of the client served and the options available in each situation.

Another issue with real-time systems is reliability. Data servers and LAN/WAN networks have become critical to the operation of many businesses today, including utilities. Real-time modeling systems have the potential to become business-critical as well. The standard for reliable operation is much higher for a continuously running system than for a desktop application, so it will require appropriate staff to keep it functioning properly. Real-time modeling systems need to have sound strategies for dealing with unexpected faults, power failure, infeasible modeling conditions, etc. Companies also need to maintain the ability to operate safely when the real-time system is not functioning.

The value of the real-time modeling system is linked not only to the reliability and completeness of the SCADA data, but also to the accuracy of its results, presented as alarms, monitors, and other displays. The model must be maintained and periodically tuned or recalibrated to be sure its predictions are still acceptable. It is helpful to build in "intelligence" to remove alarms, etc., which might be based upon erroneous inputs and modeling results. Users should be trained to understand that the modeling-based outputs are not direct measurements themselves. Experience in the pipeline industry has shown that, after a "proof period," operators can come to strongly trust real-time modeling systems [7]. When the above issues are dealt with reasonably, real-time modeling systems offer substantial benefits.

CONCLUSIONS

The delivery of drinking water is becoming more constrained by safety, cost, regulatory, and security concerns. Operational equipment and configurations are becoming more sophisticated, while operations staff may be less experienced today due to increased employee turnover and many recent retirements. These and other concerns point to a need for better operational decision support.

SCADA systems and hydraulic and water quality modeling systems are prevalent and well accepted. By joining these together, it is possible to provide operators, managers, and engineers with tools that support sound decision making for the day-to-day operation of the distribution system. A vision for real-time monitoring and modeling has been presented. It is believed that this system will support better understanding of system operations and therefore provide more confidence for current and future configurations and operational strategies, resulting in safer and more reliable delivery of drinking water.

ACKNOWLEDGMENTS

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CHAPTER · 26

Development of an On-line System to Monitor the Rate of Corrosion and Metal Release in Drinking Water

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SUMMARY

This chapter describes results of experiments and theoretical modeling relevant to the development of a new approach to on-line monitoring of metal release and corrosion. This is done by means of measurements of changes of the corrosion potential (E_{corr}) during stagnation and flow (CPSF approach). In the absence of free chlorine, the values of E_{corr} are strongly correlated with the concentration of copper. In the presence of chlorine, the behavior of E_{corr} has an additional feature: immediately following the start of a stagnation episode, the corrosion potential rapidly decreases and reaches a minimum. This type of E_{corr} behavior was examined based on numerical modeling. It was determined that the intensity of the E_{corr} transients is sensitive to the presence of chlorine and increases linearly with its concentration. It was concluded that for systems for which the behavior of E_{corr} without chlorine in CPSF cells has been explored, its quantification in the presence of chlorine can be utilized to determine the rate of free chlorine decay and the corrosion current defined by the reduction of oxygen.

INTRODUCTION

Current methods to monitor the rate of corrosion of heavy metals (e.g., copper, lead/tin solder, brass) and their release in drinking water rely on a sampling routine followed by analyses for the metals of interest or on electrochemical (EC) corrosion rate measurements (in some cases, on both). These methods and their combination constitute a powerful approach, which nonetheless has its limitations. These include the need to employ relatively sophisticated equipment for EC tests, limits of the interpretability of the EC data, possible changes of the surface's state, and the need to carry out regular sampling and analyses. [e.g., 1,2,3].

AN ALTERNATIVE APPROACH

In our ongoing study, we have attempted to develop an alternative approach for on-line corrosion and metal release monitoring. The key indicator utilized in this method is the value of the corrosion potential (E_{corr}) and its behavior in stagnation and flow episodes. The technique is accordingly termed the Corrosion Potential during Stagnation and Flow (CPSF) method. Complex phenomena that occur in the corroding system during stagnation episodes have been discussed in the literature [e.g., 4], but little effort has been made to examine their behaviors based on E_{corr} measurements. Theoretical analysis of the corrosion processes shows that, in the absence of free or combined chlorine species, the corrosion potential of a copper pipe during stagnation can be described by the following "master" equation:

$$E_{corr} = E_{corr}^{no\min al} + y_1 \log[Cu^{2+}] + y_2 \log[O_2]$$

That is, the CPSF master equation predicts the existence of a linear correlation between E_{corr} and copper concentration in the pipe (at a quasi-constant concentration of the dissolved oxygen). This correlation has been repeatedly found to exist in laboratory experiments (see Figure 1).

In the presence of chlorine species, the behavior of E_{corr} has an important feature absent in experiments with chlorine-free water qualities. Immediately following the start of a stagnation episode, the corrosion potential rapidly decreases and reaches a minimum, after which it slowly increases. This feature is shown in more detail in Figure 2, which represents the data of CPSF measurements in Bellevue and Seattle, Wash., tap waters during the first 30 minutes of stagnation.

The nature and significance of the non-monotonic behavior of the corrosion potential needs to be quantitatively explained. This is because, based on the existence of a strong linear correlation between the value of E_{corr} and logarithms of the total copper concentration in the CPSF observed in the absence of chlorine, we have suggested that measurements of E_{corr} on-line can be utilized to monitor the rate of copper release in residential plumbing. The different pattern of the behavior of E_{corr}

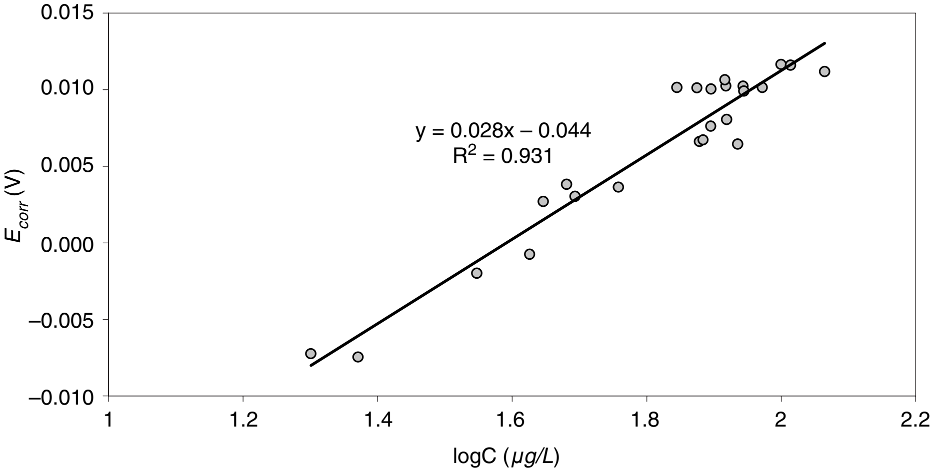

Figure 1 Correlation between copper concentration and corrosion potential values measured using the CPSF system

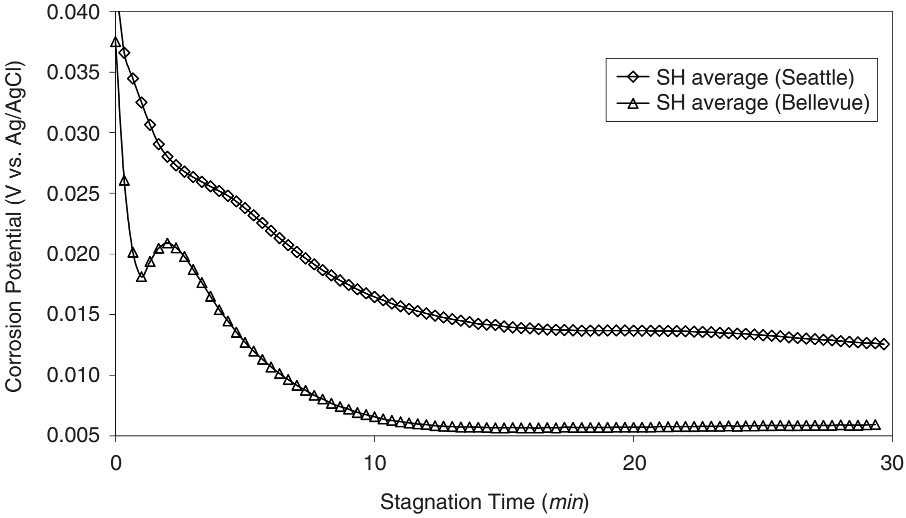

Figure 2 Changes of the corrosion potential of copper during short stagnation periods in Bellevue and Seattle tap water. Potentials are given versus Ag/AgCl standard reference electrode.

shown in Figure 2 indicates that this interpretation is not applicable in all cases, and an alternative approach in interpreting CPSF data generated needs to be developed.

To do so, we hypothesized that the initial rapid phase of E_{corr} changes is likely to be associated with corrosion reactions specific to free chlorine. Indeed, the standard EC potential of the chlorine/chloride couple is more positive than that of the oxygen/water couple (1.48 and 1.23 V versus the normal hydrogen electrode, or n.h.e.). The standard current of chlorine reduction is higher than that of oxygen and, as a result, copper release is increased in the presence of chlorine species [5,6].

To quantify the influence of the chlorine/chloride couple on the behavior of E_{corr}, we utilized POLYMATH® 5.1 software [7], which incorporates a system of differential and explicit equations that describe the consumption of oxygen and chlorine and release of copper. The program also calculates the values of E_{corr}, actual corrosion current (i_{corr}), and their changes during stagnation episodes. The numerical values used in calculations are listed in Table 1. Initial calculations indicated that there exist a range of EC rates of copper oxidation currents for which the E_{corr} transients have the magnitude and duration similar to those observed experimentally. Given this result, further modeling of the E_{corr} transients associated with the concurrent reduction of free chlorine of oxygen was carried out using a fixed value of the standard EC copper oxidation current (10^{-6} A/cm^2). (This parameter is not identical to the rate of corrosion, E_{corr}, which is a complex function of several factors, including the EC current of copper oxidation.)

The next series of calculations performed for free chlorine concentrations varying from 0 to 1.2 mg/L showed that the intensity of the transient feature in the E_{corr} versus time profiles increases almost linearly with the chlorine concentration (Figure 3). It was also determined that, due to its higher oxidative activity and in agreement with the literature [5,6], free chlorine initially enhances the rate of corrosion and increases the rate of copper release.

Due to the sensitivity of E_{corr} to the presence of free chlorine and the intrinsic association of chlorine decay with the rate of corrosion in that system, it was concluded that for systems for which the behavior of E_{corr} without chlorine is well explored, its quantification in the presence of chlorine species can be used to elucidate the intensity of the corrosion current defined, after an E_{corr} transient has disappeared, solely by the reduction of oxygen. Fundamentally, this is because the consumption of free chlorine, which can be tracked on-line through the measurements of E_{corr},

Table 1 Parameters used in the numerical modeling of corrosion processes in the CPSF cell

Parameter	Value
Standard potential of the Cu^{2+}/Cu couple*	0.34 V
Standard potential of the $HClO/Cl^-$ couple	1.48 V
Standard potential of the O_2/H_2O couple	1.23 V
Standard current of oxygen reduction	$1 \cdot 10^{-4}$ A/cm^2
Standard current of free chlorine reduction	$3 \cdot 10^{-4}$ A/cm^2
Standard current of copper oxidation	$1 \cdot 10^{-7}$ to $1 \cdot 10^{-5}$ A/cm^2
Tafel slope for all electrochemical reaction†	0.145 V^{-1}
Initial concentration of oxygen	$2.5 \cdot 10^{-4}$ M
Initial concentration of free chlorine	0 to $1.69 \cdot 10^{-5}$ M
Concentration of inorganic chloride	0.001 M
pH of hypochlorous acid	7.3
Temperature	25°C
Distance between planar electrode in the CPSF cell	0.5 cm

* All potentials in this table are quoted versus the standard normal hydrogen electrode (n.h.e.).

† All expressions used in POLYMATH employed natural exponents to calculate EC currents and their dependence versus potential.

is defined by the overall rate of corrosion, and the higher the value of i_{corr}, the more rapidly will the E_{corr} transient disappear.

The sensitivity of the shape of the transient to moderate changes of the standard EC copper oxidation current (from $8 \cdot 10^{-7}$ to $1.3 \cdot 10^{-6}$ A/cm^2) was examined based on the results on numerical modeling. It was determined that the time corresponding to a minimal value of E_{corr} (clearly observed in experiments with Seattle and Bellevue tap water, Figure 2) was notably affected by the electrochemical parameters. To examine the behavior of E_{corr} in more detail, we calculated the derivatives of E_{corr} versus time profiles, whose shape was found to closely resemble that observed experimentally (Figure 4). It was shown that the location of the minimum of E_{corr}, which corresponds to zero values of the derivatives, shifts toward lower times with the increase of the standard rate of copper oxidation.

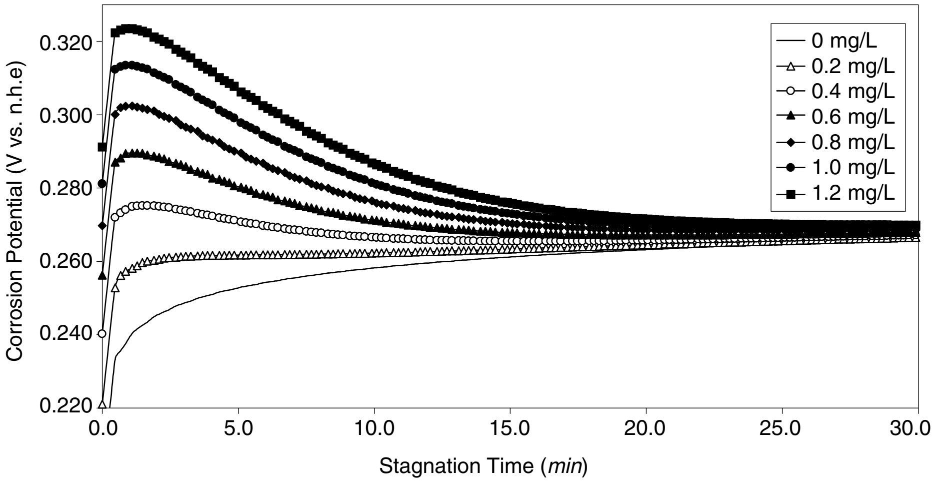

Figure 3 Modeling of the transient behavior of the electrochemical potential of copper at varying levels of free chlorine and fixed intrinsic electrochemical oxidation current of copper

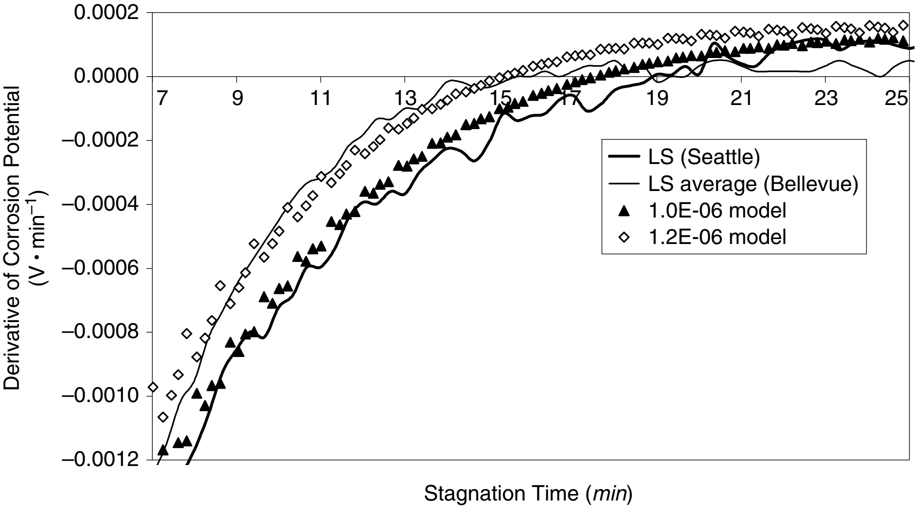

Figure 4 Comparison of experimental and calculated derivatives of E_{corr} transients

Detailed numerical analysis shows that values of the time at which the minimum of E_{corr} is observed (t_{min}) are expected to be highly sensitive to actual rates of corrosion, i_{corr}. Indeed, the modeling predicts that the change of i_{corr} values of only about 5% to 10% is likely to cause a considerable change of t_{min} (e.g., from about 20 to 15 minutes). In all likelihood, this level of sensitivity is exaggerated and the realities of experimental corrosion measurements in drinking water (e.g., electronic noise, variations of temperature, oxygen and chlorine concentrations) will tend to decrease it. Nonetheless, the above theoretical consideration yields an important conclusion that both the intensity of the transient E_{corr} feature and t_{min} values can be utilized to quantify the rate of corrosion.

CONCLUSIONS

Our research shows that in the case of CPSF measurements of E_{corr} in drinking water without chlorine, a strong linear correlation exists between E_{corr} values and logarithms of the total copper concentration. Based on this, it is suggested that on-line measurements of E_{corr} can be utilized to monitor the rate of copper release in residential plumbing. In the presence of free chlorine, an alternative approach to CPSF monitoring needs to be used because E_{corr} values exhibit more complex transient behavior. Based on numerical modeling data, it was determined that the intensity of the E_{corr} transients is likely to increase linearly the concentration of free chlorine, but the rate of its decay is defined solely by electrochemical properties of the corroding metal. As a result, for systems for which the behavior of E_{corr} without chlorine in CPSF cells has been defined, measurements of E_{corr} transients can be used to determine the values of corrosion current.

ACKNOWLEDGMENTS

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CHAPTER · 27

New Analytical Tools for Distribution System Surveillance

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SUMMARY

The presence of coliforms in distribution system samples is one of the major concerns for utilities. Progress has to be made in the area of coliform analysis to allow utilities to improve their control of distributed water quality. The DVC-FISH method and an enzymatic ChemScan®*RDI*-based method are two new analytical methods. After evaluation, they proved to be more sensitive, specific, and rapid than the standardized culture method and to be well adapted to monitor the distribution system contamination.

The study presented here covers a survey in the suburbs of the Paris network. Parameters under study were *E. coli* counts using the DVC-FISH technique, the ChemScan®*RDI* technique, and the standardized culture method.

The results demonstrate that the water quality was excellent during the study, with very little noncompliance and no confirmed coliform-positive sample in the official sanitary control (over 100 samples/week). In spite of these satisfactory results for compliance, the use of the DVC-FISH technique revealed that *E. coli* were nevertheless present in the distribution system, although not detectable with agar cultures. When comparing *E. coli* counts obtained with the DVC-FISH and the ChemScan®*RDI* technique, it appears that the DVC-FISH technique leads to a higher number of positive results than the ChemScan®*RDI* technique.

The spatial distribution of positive samples appears to be uneven, with four points that were never positive and two that were positive in 47% of cases. The use of this new analytical technique proved to be very

promising and to give appreciably better insight into water quality. One of the major problems for utilities is the significance of coliform-positive samples, and these new techniques obviously are an alternative to current bacteria counts. There is, therefore, good potential for application in drinking water monitoring, allowing utilities to anticipate noncompliance and take protective measures in time.

INTRODUCTION

The presence of coliforms in distribution system samples is one of the major concerns for utilities. Coliforms can either break through the treatment barriers or regrow in the distribution system biofilm, resulting in noncompliance. Different studies on this latter phenomenon have shown that coliforms can be detected in the biofilm in spite of satisfactory water quality. Studying biofilm rather than bulk water samples would probably lead to a more accurate image of the coliform contamination of the distribution network. However, it is not possible to directly monitor the quality of the distribution system by sampling biofilm because the methods required for that purpose (coupons) are very tedious and long. Therefore, the best solution seems to rely on a dual approach: (1) to conduct research programs aiming at understanding the mechanisms involved in the growth of coliforms in biofilms, and (2) to monitor the quality of the distribution system by testing either a high number of water samples or sensitive methods such as the DVC-FISH technique.

In the case of SEDIF and its operator, which supplies water to nearly 4 million inhabitants in the suburbs of Paris, water is abstracted from three rivers and then treated using a biological treatment including ozone disinfection, granular activated carbon filtration, and final chlorination. On the SEDIF networks, the percentage of noncompliant water samples is lower than 0.5% per year based on the results of around 8,000 samples. These results are statistically significant due to the high number of samples. At the week scale, the results of only 100 samples are available per water origin, making the interpretation of the data more difficult. This is because the low number of samples and the lack of sensitivity of the analytical culture methods do not yield statistically significant results in terms of noncompliance. In order to improve this control, the development of alternative techniques is an important challenge. The requirements of these new techniques are an improved sensitivity and specificity of the detection and a decrease in the time required to obtain the results.

FISH (Fluorescent *In Situ* Hybridization) is a molecular detection method based on the hybridization of a fluorescently labeled probe on a specific genetic sequence of the targeted microorganism (Amann et al., 1995). In general, the specific sequences of the target are located within the 16S rRNA. After microscopic observation, the presence of the target microorganisms can be visualized and quantified. This method has been used for the detection of different types of bacteria or parasites. Like all the molecular techniques, the FISH method is very sensitive, specific, and rapid. This method, however, does not allow a direct determination of the viability of the detected microorganisms. The viability concept is particularly fundamental when disinfected water samples are analyzed. It is therefore necessary to associate a viability test, such as DVC (Direct Viable Count) for the detection of viable bacteria, with the FISH method. The DVC relies on pre-incubation of the bacteria in a resuscitation medium containing an antibiotic cocktail (Kogure et al., 1979). During this step, the cellular machinery of viable bacteria is stimulated, but the cellular division is inhibited by antibiotics. This results in the elongation of the viable bacteria, a phenomenon that can be visualized after the FISH labeling. The delay of analysis is dependent on the type of targeted bacteria. A protocol of DVC-FISH has been developed for the specific detection of *Escherichia coli* (Régnault et al., 2000), and results can be obtained within the day of sample reception.

Another detection method combines a novel two-step enzymatic labeling technology with laser-scanning cytometry (ChemScan®*RDI*), (Nelis & Van Poucke, 2000; Van Poucke & Nelis, 2000a, 2000b). Enzymatic labeling technology using β-glucuronidase as a marker enzyme allows one to selectively label active *E. coli* cells after membrane filtration of a 100-mL water sample. After laser-scanning of the entire 25-mm membrane filter, a scan map locates the *E. coli* cells. These cells can subsequently be visualized thanks to an automated epifluorescence microscope driven by the ChemScan®*RDI*. The results are obtained within four hours. This method has been validated in our laboratory, and equivalent or higher *E. coli* counts than with culture techniques have been obtained (unpublished results).

The aims of this study were: (1) to evaluate the DVC-FISH detection method by comparison with the standard method for the detection of *E. coli* from water samples, and (2) to apply the new methods (DVC-FISH, ChemScan®*RDI* detection) to a survey of the Paris suburbs distribution network, in order to assess their efficiency in improving the statistical significance of the microbiological control of distributed water.

MATERIALS AND METHODS

Sampling

Water samples were taken from two different sectors (named networks A and B) of the Paris suburb network. The network is fed by two surface water treatment plants using coagulation, flocculation, settling, rapid sand filtration, ozonation, biological filtration on granular activated carbon, and chlorination processes. In each sector, eight sampling points were selected, some of them because noncompliant results had been obtained in the past. Sampling at a weekly frequency began in April 2000 and ended in August 2000. A total of 126 samples were collected. Each sample was collected in a sterile flask containing sodium thiosulfate. Samples were maintained at 4°C for a maximum of 24 hr before being analyzed for *E. coli* by DVC-FISH, the ChemScan®*RDI* method, or the standardized culture method. Parallel to this specific sampling, the official sanitary control using standard methods was performed by CRECEP (Centre de Recherche de Contrôle de la Ville de Paris) on networks A and B.

Enumeration of Total and Fecal Coliforms and *E. coli* Using Standardized Methods

The analyses were performed according to the AFNOR standard method (NF T 90-414). Of each sample, 100 mL were filtered on a cellulose membrane (porosity = 0.45 µm). The membranes were then incubated directly on selective TTC-tergitol lactose agar medium 48 hr at 37°C for total coliforms and at 44°C for fecal coliforms. After the incubation period, the typical lactose-fermenting colonies were confirmed using the oxydase test for total coliforms and the indole test for *E. coli* identification.

Enumeration of *E. coli* Using DVC-FISH

One liter of sample was filtered on polycarbonate HTTP membrane (porosity = 0.4 µm). The membranes were then incubated on a pad saturated with trypto-casein-soya (30 g/L) medium supplemented with yeast extract (6 g/L) and two types of antibiotics: nalidixic acid (10 µg/mL) and ciprofloxacin (1 µg/mL). The incubation period was four hours. The membranes were then used for FISH labeling as previously described (Regnault et al., 2000). Elongated *E. coli* cells were enumerated using an epifluorescence microscope: for each sample, a minimum of 100 microscopic fields (out of about 12,000 for the entire membrane) were observed.

Enumeration of *E. coli* Using the Enzymatic Method, Referred to as the ChemScan®*RDI* Technique

The analysis was carried out using the appropriate kit (Chemunex, France). Briefly, 100 mL of sample was filtered through a membrane. The labeling step was then performed directly on the membrane by incubation with a nonfluorescent fluorescein derivative. This compound is the substrate of the β-glucuronidase, and it is cleaved if viable *E. coli* with active enzyme and intact cellular membrane are present, thus rendering the bacteria fluorescent. Finally, the membranes were placed in the ChemScan®*RDI* analyzer, where in three minutes, the laser beam scanned the entire surface for fluorescent signals. The results were visually confirmed by an operator using epifluorescent microscopy.

RESULTS AND DISCUSSION

Validation of DVC-FISH Method Using Disinfected Water Samples

FISH is a very specific and sensitive analytical method that allows rapid identification and enumeration of bacteria from water samples. Because the viability of the detected bacteria cannot be directly determined, it is necessary to associate a viability assay with the FISH test. In collaboration with the Institut Pasteur de Paris, we have developed a DVC-FISH approach applied to the detection of *E. coli* from water samples. The DVC step allows the elongation of viable bacteria and the FISH step specifically identifies *E. coli*.

This detection method was first validated using laboratory-grown environmental *E. coli* strains. With strains from exponential phase cultures, equivalent counts have been obtained by DVC-FISH and the culture method (culture at 44°C followed by indole test) (unpublished results). The second part of the validation was carried out using different types of naturally contaminated water samples (raw water, partially treated, or potable water samples). Figure 1 displays the results obtained with 16 samples of potable water analyzed in parallel using DVC-FISH and the standard method. All the samples were negative for *E. coli* using the latter method, whereas *E. coli* were detected in all the samples analyzed using DVC-FISH, with contamination levels ranging from 1.19 to 11.9 *E. coli*/100 mL.

These results illustrate the higher sensitivity of the molecular detection method compared with the culture method, especially when stressed environmental bacteria are present. Therefore, DVC-FISH seems

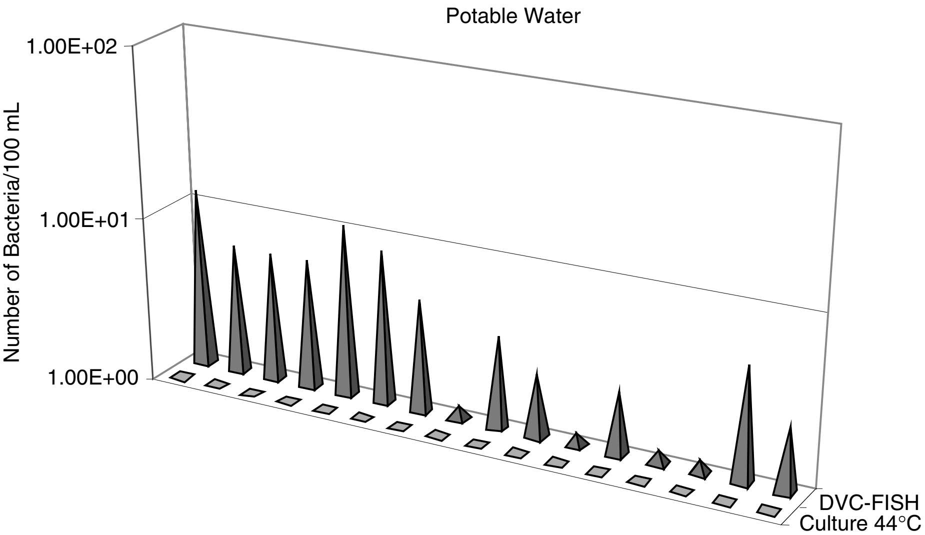

Figure 1 Detection of *E. coli* in potable water samples by DVC-FISH and culture methods

to be an excellent tool for the evaluation of the contamination level of distribution networks.

Network Study

Networks A and B are located in the suburbs of Paris. The water from these networks undergoes two types of controls: the official sanitary control performed by the CRECEP, and the surveillance by Vivendi Water Central Laboratory, resulting in over 100 samples per water origin analyzed each week.

However, two limitations exist for an accurate assessment of water quality on the basis of these results: (1) the proportion of positive results is low for these positive samples, and (2) the number of bacteria is very close to the detection limit of the method. In these conditions, it is difficult to determine whether the presence of coliforms is statistically significant or not. The development of more sensitive analytical methods could therefore be an opportunity to improve the significance of the results and give appreciably better insight into water quality.

Water quality during the period of the study based on the data obtained with the official sanitary control

The results of the official sanitary control obtained during the period of the survey are shown in Tables 1 and 2 for the two networks. These results illustrate the high quality of the water in the two networks during the entire survey.

Comparison of the results obtained with the three analytical methods

All the samples have been analyzed in our laboratory using the standardized culture method, the ChemScan®*RDI* method, and DVC-FISH for the detection of *E. coli*. The results for the two networks are presented in Table 3. While *E. coli* was never detected by the standard method, positive results were obtained using the ChemScan®*RDI* and DVC-FISH methods. *E. coli* was detected by DVC-FISH in 19% and 13% of cases for networks A and B, respectively. The confidence interval was 11% to 30% for network A, and 6% to 23% for network B ($\alpha = 0.05$).

Table 1 Results of the official sanitary control on network A

	Detection of Fecal Coliforms	Detection of Total Coliforms	Detection of Enterococci
Mean (%)	0.2	0.5	0.0
Median (%)	0.0	0.0	0.0
STD (%)	4.9	7.0	0.0
Number of samples	819	819	819

Table 2 Results of the official sanitary control on network B

	Detection of Fecal Coliforms	Detection of Total Coliforms	Detection of Enterococci
Mean (%)	0.2	0.5	0
Median (%)	0.0	0.0	0
STD (%)	4.7	7.4	0
Number of samples	905	905	905

Table 3 Detection of *E. coli* by the standardized and alternative methods

	Number of *E. coli* Detected			Total
	0	1	2	
Culture method	119	0	0	119
ChemScan®*RDI* method	116	10	0	126
DVC-FISH method	106	16	4	126

These results show the improved sensitivity of the two alternative detection methods compared with the culture method. This higher sensitivity can be explained by the ability of the molecular or enzymatic methods to detect a bacterial activity that can be stimulated during the experiment, even if the bacteria have been stressed by the previous treatments. These activities could be stimulated and detected even though the culturability could not be recovered, as shown by the negative results obtained with the standard method.

In this study, the DVC-FISH method seemed more sensitive than the ChemScan®*RDI* method, since a higher number of samples were positive with the former than with the latter. Moreover, 90% of ChemScan®*RDI*-positive samples were also positive using DVC-FISH, whereas 45% of positive DVC-FISH samples were positive using the ChemScan®*RDI* method. The Spearman correlation coefficient (R) between DVC-FISH and ChemScan®*RDI* counts was significant ($R=0.6$, $p<0.0001$). Although the two methods target different physiological bacterial activities, their results are well correlated, thus demonstrating the pertinence of these techniques.

Relationship between the official sanitary control and the DVC-FISH results

Figures 2 and 3 show the weekly detection rate of fecal coliforms by the official sanitary control (standard method) and of *E. coli* by DVC-FISH for networks A and B during weeks 18 to 28.

Although the two types of controls were not carried out on the same samples, we found it interesting to compare the results in terms of "positive or negative weeks," meaning that positive samples had or had not been observed. No temporal relationship can be observed between the percentage of positive samples using DVC-FISH for the two networks. In the survey, noncompliant results appeared during two weeks for each network based on the official sanitary control. For network A, the

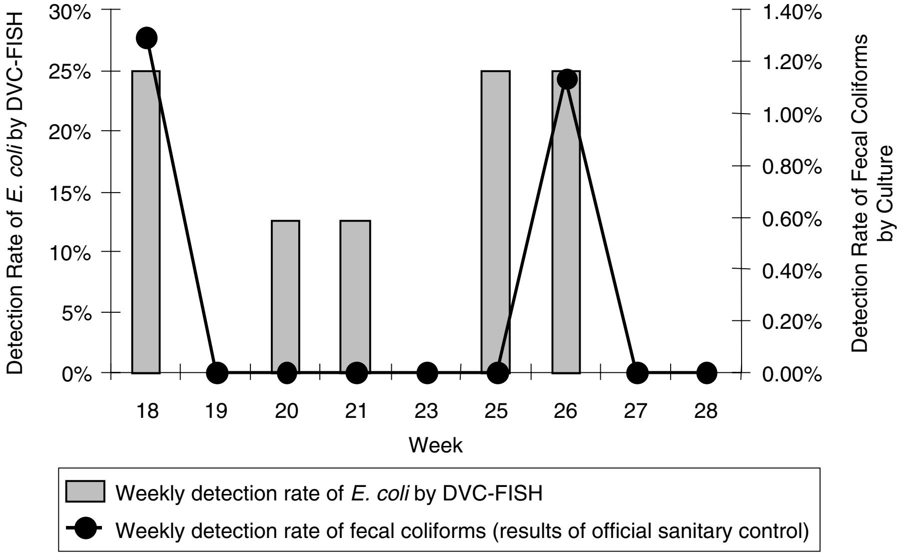

Figure 2 Weekly detection rates of *E. coli* and fecal coliforms in network A

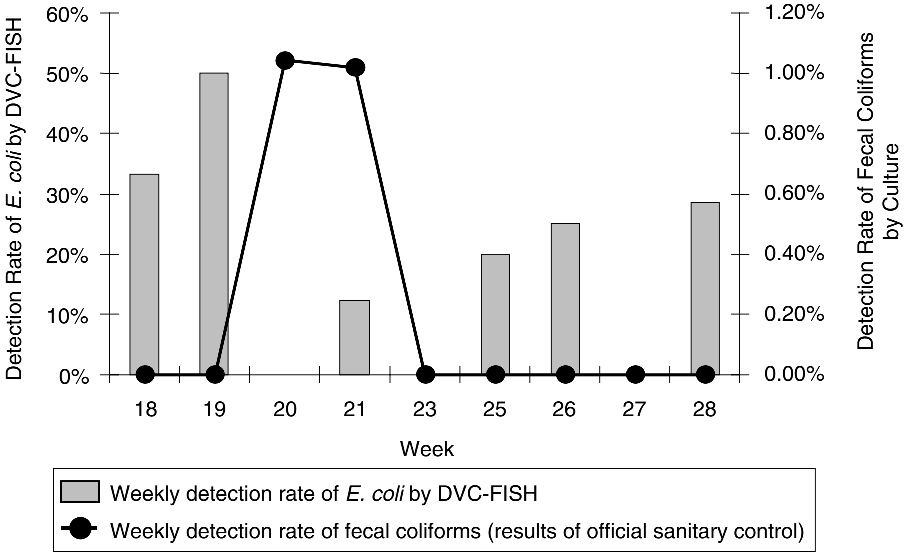

Figure 3 Weekly detection rates of *E. coli* and fecal coliforms in network B

DVC-FISH analysis also resulted in a number of positive samples during the same two weeks. For network B, positive samples were found with both methods during week 21. However, during week 20, some positive results were found by culture, whereas all the samples were negative by DVC-FISH. One possible explanation for this discrepancy is that fecal coliforms (not necessarily *E. coli*) are detected in the official control, whereas *E. coli* is targeted by the DVC-FISH method.

Spatial distribution of positive samples by DVC-FISH

In Figure 4, the spatial distribution of positive samples is shown. Whatever the network, a nonrandom spatial distribution of positive samples can be observed. In each network, certain sampling points were always negative while the frequency of *E. coli* detection for other sampling points (namely A8 and B8) was high (50%). This observation confirms the significance of the positive results obtained by the DVC-FISH method.

CONCLUSIONS

This work proved that the use of new analytical methods such as the DVC-FISH or ChemScan®*RDI* techniques for the detection of *E. coli* in distribution systems can be of interest because of the higher sensitivity of

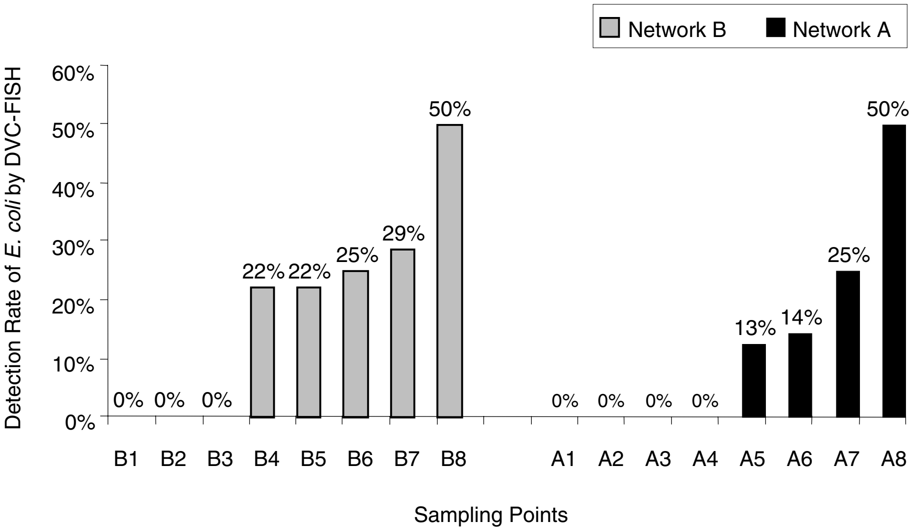

Figure 4 Spatial distribution of positive samples by DVC-FISH

these methods compared with the standard culture ones. Another advantage of these new techniques is the decreased time between sample collection and availability of results (one day versus two to three days for the traditional methods).

For the microbiological control of distribution networks, the traditional methods for fecal coliforms lead to low percentages of positive samples. Therefore the significance of these percentages can be questioned. The increased sensitivity of the new techniques might overcome this limitation by reaching a higher number of positive samples. Additional work is needed in order to study in more detail the relationship between the results obtained with these techniques, the culture methods, and the sanitary quality of the water.

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CHAPTER · 28

Rapid Analytical Techniques for Drinking Water Security Investigations

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BACKGROUND

More than two years after the terrorist attacks of Sept. 11, 2001, concerns about future attacks continue to draw considerable attention and demand significant resources. These increased security concerns center on state-sponsored and international terrorist groups as well as radical domestic groups and individual extremists. Public water supplies are included on the list of potential targets for malevolent acts because they directly affect large numbers of people and can be difficult to protect. Water system security concerns focus on the possibility of threatened or actual contamination of drinking water because this scenario could have a rapid and deleterious effect on public health.

GUIDELINES DETAIL UTILITY RESPONSE TO THREATS

The US Environmental Protection Agency (USEPA) has been designated the lead federal agency for protection of the nation's drinking water and wastewater infrastructure and has formed the Water Security Division to carry out this responsibility. In early 2002, USEPA published preliminary guidance for water and wastewater systems on response, recovery, and remediation actions for malevolent acts directed toward utilities (USEPA, 2002).

The agency is in the process of completing detailed guidance (Response Protocol Toolbox) for drinking water utilities to follow in response to threatened or actual incidents of intentional contamination of public water systems (USEPA, 2003). The guidance suggests specific steps that should be taken by response teams performing the initial investigation ("site characterization") of the site of suspected contamination. As described in the protocol, under certain circumstances (i.e., in the absence of evidence of acutely hazardous conditions), it would be appropriate for water utility personnel to perform the initial site characterization and sampling as part of the response. Site characterization by a utility team is especially appropriate in situations involving security breaches, such as cut fences or locks. Whereas once these activities might have been dismissed as the work of vandals, they now must be investigated further to ensure that more serious acts have not occurred.

Team Conducts Initial Screening

The site characterization team is encouraged to perform a field safety screening of environmental surfaces and air to detect acute hazards at the suspected contamination site that might pose an exposure risk to site personnel. Additionally, the investigation team is directed to conduct rapid field testing of the water. This water analysis is also intended to detect acute hazards for the site team (and subsequently for lab analysts) as well as to provide "presumptive" data on the nature of contaminants that may be present in the water. These presumptive data may help guide the utility's operational and public health responses that might need to be taken before definitive laboratory analytical results become available. The field-screening tests recommended by USEPA include radioactivity measurements of air and water as well as testing of water for basic parameters such as pH, chlorine content, and cyanide. According to the guidance, these core tests should be supplemented by additional field testing commensurate with the resources of the site characterization team.

Research Assessed Analytical Methods

A previous article detailed efforts by the Pittsburgh (Pa.) Water and Sewer Authority (PWSA) and the University of Pittsburgh School of Engineering to evaluate analytical methods that can be used by water utility laboratories in response to a threat, in the event of suspected tampering with a water system, and during periods of heightened security (States et al., 2003). In this chapter, the authors describe commercially available rapid analytical technologies (some of which have become available only recently) that could be used by utility personnel or emergency response teams to generate preliminary data during the site characterization phase of a suspected intentional contamination event. Some of these techniques have been evaluated by PWSA and the University of Pittsburgh to help determine their applicability to this task; the data resulting from this process are included here. The reviewed technologies include:

- rapid immunoassays
- rapid enzyme tests
- rapid polymerase chain reaction (PCR) techniques
- field-deployable gas chromatography/mass spectrometry (GC/MS)
- acute toxicity screening methods

The specific equipment brands and manufacturers described in this chapter do not represent all of the technology currently available. These results and discussion spotlight devices that have been acquired by PWSA during the past year and are currently used in the utility's security monitoring program.

METHODS EMPHASIZE SPEED AND PORTABILITY

Rapid Immunoassays Are Immunoassay-based Identification Systems for Field Screening

These immunochromatographic assays qualitatively detect chemical or biological agents (antigens) by relying on the specificity of an antigen–antibody binding event. Immunoassay technology has been used as an analytical tool since the early 1980s to detect pathogens in clinical specimens and since the 1990s in environmental applications to detect pesticides. When used to screen for biological agents, the antibodies target proteins that are unique to the agent. Several kits are commercially available for on-site screening of specific chemical and biological substances that might be used to intentionally contaminate air, food, or water. Results are considered presumptive and are generally available within 15 min.

Rapid on-site screening of air and water samples for volatile organic compounds can be conducted with portable purge-and-trap gas chromatography/mass spectrometry units, such as this one from INFICON.

Test strips offer ease of use

BioThreat Alert® test strips[*] are based on the analytical technique of lateral flow immunochromatography. The suspect material (solid or liquid) is mixed with buffer, and five drops of the mixture are then added to the sample port of the test strip. The target agent moves by capillary action along the membrane inside the test strip's plastic case. If the level of target substance in the sample is present above a certain concentration, the antibodies and target antigen combine in the test strip to form a reddish band that appears in a window. If a band appears only in the "C" window, the test is negative. This result indicates a successful positive control utilizing a control antigen provided in the kit. If a band also appears in the "S" window, the sample is positive for the agent of concern.

The test strips can be read visually without the use of instrumentation. However, an electronic test strip–reading device[†] is also available. According to the manufacturer, the reader decreases the detection limit by 0.5- to 1-log units because the optical technology can recognize faint positive results that may be difficult to see with ambient lighting. The reader is portable, powered by alternating current or battery, and can be taken to the site of suspected contamination. Related testing tools include swab-collection kits for sampling environmental surfaces and a battery-

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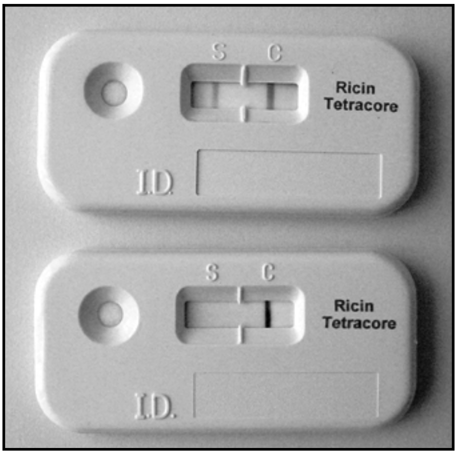
[*] Tetracore Inc., Gaithersburg, Md.

[†] Guardian Reader, Alexeter Technologies LLC, Wheeling, Ill.



If the level of a target substance is present above a certain concentration, the antibodies and target antigen combine in the rapid immunoassay test strip to form a reddish band that appears in a window. If a band appears in both the “C” and “S” windows, the test is positive for the agent of concern. If a band appears only in the “C” window, the test is negative.

operated portable air sampler* that collects and concentrates aerosol samples and adds the sampled particles directly to the test strip. Screening of environmental surfaces, air samples, and water at the site of suspected contamination is consistent with the field safety screening and rapid water testing recommendations outlined in USEPA’s Response Protocol Toolbox.

Currently the manufacturer of the test strips offers the following strips with the detection limits specified; detection limits were obtained without the use of an electronic reader:

- *Bacillus anthracis* (anthrax)—1×10^5 cfu/mL
- *Yersinia pestis* (plague)—2×10^5 cfu/mL
- *Francisella tularensis* (tularemia)—1.4×10^5 cfu/mL
- *Clostridium botulinum* (botulism) toxin—10 µg/L
- staphylococcal enterotoxin B (SEB)—2.5 µg/L
- ricin—50 µg/L

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* BioCapture BT-550, Mesosystems Technology Inc., Albuquerque, N.M.

Similar lateral-flow immunoassay kit also detects and identifies multiple analytes

The SMART* (Sensitive Membrane Antigen Rapid Test) system detects agents (antigens) in the sample by immunofocusing colloidal gold-labeled reagents (antibodies) and their corresponding target antigens onto small membranes. A 100 µL (approximately three-drop) liquid sample is added to the test ticket; if red lines form in both the internal control and sample compartments within 15 min, a positive result is indicated. Tests are commercially available for anthrax, cholera, tularemia, and plague bacteria as well as *C. botulinum*, ricin, and SEB toxins.

Unlike the BioThreat Alert® test strips, the SMART system does not have an electronic reading device. However, the manufacturer does sell accessory kits to assist in collecting samples from liquids, air, and environmental surfaces. The overall detection limit claimed for the product is similar to that of the BioThreat Alert® test strips, i.e., 10^5 cfu/mL for bacteria and 50 µg/L for biotoxins.

Sensitivity is an issue for rapid immunoassays. Although rapid immunoassays are merely a presence–absence screen for a small number of select contaminants, they may be especially useful for site characterization teams trying to detect high-level hazards present at a sampling site prior to sample collection. The sensitivity of the tests is a critical issue surrounding the use of rapid immunoassays for presumptive screening in drinking water security investigations. Manufacturers claim a detection limit of 10^5 cfu/mL for anthrax spores. King and co-workers (2003) conducted a comparative study of three commercially available anthrax immunoassays and obtained results similar to these claims without the use of instrumental readers.

This detection limit indicates that the minimum concentration of anthrax spores these assays could detect in water is 100,000 spores/mL. According to the manufacturers, these detection levels could be reduced by 0.5 to 1 log through the use of a commercially available instrumental reader, yielding at best a detection limit of 10,000 spores/mL. Such relatively high concentrations suggest a limited usefulness for the assays in detecting traces of agents in water systems in which the contaminants may have been added hours earlier and may already have been diluted by the municipal system. However, use of ultrafiltration in the field could potentially concentrate these samples 400-fold. Research is currently

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* New Horizons Diagnostics, Columbia, Md.

under way in several utility, government, and academic laboratories to develop a tangential-flow, hollow-fiber ultrafiltration technique that could concentrate 100 L of sample water to a 250-mL concentrate in the field. Assuming good recovery of agent from the filters and no parallel concentration of method interferences, these concentration measures could reduce the detection limit of the immunoassay technique to 25 cfu/mL (at least for bacterial cells but perhaps not for biotoxins, which are smaller macromolecules). Although the test is still relatively insensitive, its simplicity and rapidity could make it useful at least for field safety screening purposes, e.g., detecting gross contamination that might pose an acute risk to site investigators and sample collectors. Theoretically, detection limits could be lowered even further by subsequent centrifugation of the field concentrate in a local laboratory. Studies are needed to determine the actual detection limits achievable using these approaches.

Specificity is another concern with field use of rapid immunoassays

The usefulness of rapid immunoassays would be severely limited by cross-reactivity of the tests with other, possibly nonpathogenic microorganisms, which could lead to false-positive results. Independently acquired quality control data are limited. However, one manufacturer has claimed a false-positive rate of 3% for *B. anthracis* (anthrax) determinations in tests that included 30 different non-anthrax bacterial species, whereas false-positive rates for competing products range from 37% to 83%. The same manufacturer claimed a false-positive rate of 7% for *Y. pestis* (plague) tests in studies involving 15 different nonplague bacterial species. Independent evaluations of the potential for cross-reactivity for these assays are needed.

Rapid enzyme tests aid in detecting nerve agents and pesticides

The nerve agent tabun was originally developed in Germany before World War II from research on organophosphate pesticides. Since that time, a number of countries have developed nerve agents in addition to tabun (e.g., sarin, soman, and VX) for delivery by the aerosol route. Nerve agents are usually dismissed as credible agents for attacks on water supplies because they tend to hydrolyze spontaneously in water, rendering them harmless. However, historical accounts state that the Ceausescu regime in Romania caused injuries during the 1989 civil uprising by intentionally contaminating municipal water storage tanks with sarin and VX.

Some pesticides (i.e., organophosphates and carbamates) and nerve agents cause adverse health effects on humans and animals by inhibiting the natural enzyme cholinesterase. When nerve impulses reach certain neuromuscular junctions in the body, the neurotransmitter acetylcholine is released. Acetylcholine excites muscle fibers, resulting in muscular contractions. Several milliseconds following the release of acetylcholine, the body typically releases the enzyme cholinesterase, which destroys excess acetylcholine and prevents reexcitation of the muscle fiber. By inactivating cholinesterase, nerve agents and some pesticides interfere with this natural control process. Acetylcholine then accumulates, with successive nerve impulses causing repeated muscular contractions (spasms), convulsions, and even death.

A rapid field enzyme test was developed a number of years ago for the qualitative detection of pesticides and nerve agents based on their inhibition of cholinesterase. In a commercially available assay,[*] a membrane disk, which had been saturated with cholinesterase and attached to a test ticket, is dipped for a period of 1 minute into a water sample. The ticket is subsequently folded for a period of 3 minutes so that the sample disk is pressed tightly against a second disk containing an ester. If no pesticides or nerve agents are present in the water sample, the cholinesterase hydrolyzes the bound ester, forming a blue-colored reaction product. However, if sufficient pesticide or nerve agent is present in the sample tested, it inhibits the cholinesterase chemically bonded to the ticket, preventing the reaction and producing no color change. A white color on the second disk indicates a positive result for the presence of pesticides or nerve agents.

Rapid field screening for the presence of pesticides and nerve agents is consistent with the site characterization procedures recommended by USEPA in its Response Protocol Toolbox. The manufacturer of the enzyme assay provides no detection data for nerve agents but lists detection limits for several pesticide groups, including carbamates (0.1–5 mg/L), thiophosphates (0.5–5 mg/L), and organophosphates (1–5 mg/L).

Rapid PCR is a nucleic acid–based molecular biology technique

Nucleic acid–based techniques detect and identify biological agents by targeting nucleic acids, i.e., deoxyribonucleic acid or ribonucleic acid (DNA or RNA). Developed in the early 1980s to amplify DNA and

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[*] Severn Trent Services, Oxfordshire, England

originally applied as a clinical analysis method, PCR has since been used extensively in genetic research and forensic applications and is becoming more widely used in environmental analysis. PCR is a very specific and sensitive analytical approach for detecting and identifying biological materials in environmental samples. Although usually not prone to false-positives, the technique is susceptible to interferences that may be present in the sample matrix.

As a tool for identification of environmental microorganisms, PCR involves manipulation of a sample to lyse microbial cells and expose DNA. This procedure can involve the use of lytic enzymes, freeze–thaw cycles, or bead-beating techniques. A combination of DNA primers, enzymes, and nucleotide bases is then added to the preparation. The primers are designed to complement and combine with a unique segment of DNA in the target microorganism (i.e., the specific virus, bacterium, or protozoan the analyst is attempting to detect). The enzyme (DNA polymerase) and excess nucleotide bases are added to synthesize copies of the target DNA beginning with the typical 25–35 base-pair primer unit.

The process of separating (denaturing) double-stranded DNA, attaching (annealing) the primer to the single-stranded target DNA, and synthesizing (extending) a new strand of DNA is controlled by cycles of temperature change (heating and cooling). Denaturation usually occurs at 94°C, annealing at 60°C, and DNA elongation at 72°C. There is a doubling of double-stranded DNA with each temperature cycle. Therefore, an original double-stranded DNA in a sample, exponentially multiplying throughout a series of 30 temperature cycles, can produce more than a billion new copies of DNA.

The presence of a large number of copies permits detection by one of several methods, such as fluorogenic gene probes and gel electrophoresis. Detection of amplified DNA can be used to identify microorganisms that carry the DNA within them. This assay can also be used to detect residual DNA that is bound to toxin preparations, such as *botulinum* toxin, if the sample containing the toxin has not been subjected to conditions that cause extensive degradation or removal of the contaminant DNA. Therefore, the method may be used to indirectly detect some biotoxins. Detection strategies can even be designed so that multiple locations within the DNA of a biological threat agent are targeted, thereby increasing confidence in the identification.

New device detects numerous bacteria that could be used as agents of bioterrorism

The R.A.P.I.D.* (Ruggedized Advanced Pathogen Identification Device) PCR system has been developed over the past several years to detect various pathogenic bacteria in clinical and environmental samples. The pathogens currently detectable by the commercially available system include *B. anthracis*, *Campylobacter* spp., *F. tularensis*, *Escherichia coli* 0157, *Y. pestis*, *Salmonella* spp., *Brucella* spp., *Listeria monocytogenes*, and *C. botulinum* toxin. The manufacturer is currently developing an additional test for *Cryptosporidium* protozoa. The R.A.P.I.D. is being used extensively by the military, but the system's speed of analysis and detection capabilities also suggest a potential application for water utility personnel or emergency response teams investigating suspected incidents of intentional contamination of drinking water.

The system process specifically involves a preliminary DNA extraction step in which the cell, or spore, walls of microbes in a small volume of water sample (or possibly a sample concentrate) are disrupted by a mechanical bead-beating process. Substances such as environmental humic and fulvic acids that could potentially interfere with the PCR process are removed using a purification kit supplied by the manufacturer. The purified sample DNA is then added to a freeze-dried mixture of DNA primers, nucleotide bases, polymerase enzyme, buffer, and gene probes. The probes, which are DNA-specific, have fluorescent dyes attached to them to permit detection of the target DNA once it is copied. A small volume of sample is then added to thin capillary tubes in preparation for DNA amplification (multiplication) by temperature cycling. Because temperature change in this system is accomplished by air heating and cooling, cycling is rapid. Forty-five cycles of temperature change can be accomplished within a 30-min period. As DNA is amplified via temperature cycling, the amount of fluorescent probe taken up by the copied DNA continues to increase. The increasing fluorescence is measured by a fluorimeter on a real-time basis during amplification, indicating the presence of target DNA as it is being synthesized.

This PCR system offers a number of advantages, e.g., the sample preparation procedure is standardized, the analytical system is closed to reduce the potential for laboratory DNA contamination, the kit includes positive and negative DNA controls, and the initial data interpretation is

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* Idaho Technology Inc., Salt Lake City, Utah

automated. The system can be used to detect pathogens and some toxins in water samples, and through the use of swabs can also detect these substances on environmental surfaces at the suspected site of contamination. Both types of screening are recommended in the USEPA response protocol for investigating suspected intentional contamination sites. A key benefit of using this system for investigation of suspected contamination events is that the system permits screening for “select” bioterrorism agents, such as *B. anthracis*, *F. tularensis, Y. pestis*, and *Brucella* spp., which can legally be cultured in the United States only by certified Laboratory Response Network labs.

According to the manufacturer, the R.A.P.I.D. system is able to reliably detect as few as 100 femtograms of amplifiable DNA material from any of the microorganisms of interest in a single analysis (e.g., in 20 µL of a water sample). This equates to approximately 20 copies of bacterial DNA, suggesting a detection limit of approximately 10^3 cfu/mL. Therefore, an important consideration at this time is development of a suitable field sample concentration technique, possibly followed by local laboratory centrifugation of the field concentrate to increase the sensitivity of the analysis to make it more effective for analysis of water samples.

Although select agents such as *B. anthracis* and *Y. pestis* were not available in the PWSA lab to evaluate the effectiveness of the R.A.P.I.D. assay, stock cultures of *Salmonella* spp. and *E. coli* 0157 were tested. These pathogens were easily detected and identified using the R.A.P.I.D. system. Additionally, cultures of *Salmonella* isolated from a known positive environmental sample of duck feces collected from an open reservoir were assayed. Again, the R.A.P.I.D. system was able to quickly detect and identify the pathogen.

Field-deployable GC/MS provides on-site detection

Since 1997 a portable GC/MS chemical identification system has been available to identify and quantify hydrocarbons (on-site) in air, soil, and water samples. The HAPSITE* GC/MS system has been used for detection of toxic substances and chemical weapons of mass destruction by the military, United Nations inspectors, and hazardous material response teams. Recently, system capabilities were expanded by the addition of a modified “Situ Probe” purge-and-trap sampling device

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* INFICON Inc., East Syracuse, N.Y.

(formerly manufactured by another supplier[*]). With this system, on-site collection and concentration of volatile organic compounds (VOCs) are accomplished by placing the probe, which is connected to the GC/MS, into the water body being analyzed.

Because it weighs only 35 lb (16 kg), the purge-and-trap GC/MS system is field-deployable; it operates either by rechargeable battery or from a 24-V converter when external power is available. It can analyze individual samples or be operated automatically for unattended continuous water stream analysis. Typical detection limits are in the low microgram to milligram per liter range. The GC portion of the system is temperature-programmable (45°–225°C) and can detect volatile compounds with a molecular weight ranging from 45 to 300. The MS portion uses an electron multiplier detector. To facilitate identification of unknowns, the system library contains the mass spectra of 170,000 organic compounds, including those of chemical warfare agents.

The advantage of a field-portable GC/MS over use of an off-site laboratory is that analyses can be run more quickly and the results used to guide a site investigation, urgent utility operational responses, and public health responses. The system can be used to screen for VOCs in both air and water samples using the probe sampler. Monitoring of both air samples (field safety screening) and water samples (rapid field testing of water) is recommended during site characterization in the USEPA response protocol guidelines.

ACUTE TOXICITY SCREENING DETECTS RANGE OF SUBSTANCES

Acute toxicity screening tests are broad-spectrum assays designed to indicate the presence of acutely toxic substances such as industrial chemicals, chemical weapons, and biotoxins. Some of the tests involve chemical reactions whereas others are bioassays. Two commercially available tests used for toxicity screening in a number of water laboratories are Microtox[†] and Eclox.[‡] Both assays have been evaluated in the

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[*] Sentex Systems Inc., Fairfield, N.J.

[†] Strategic Diagnostics Inc., Newark, Del.

[‡] Severn Trent Services, Oxfordshire, England

PWSA laboratory. A third assay, the IQ Toxicity Test,* is also discussed because it uses a different, quite sensitive indicator (whole organism) for toxic detection.

Microtox System Can Be Used in the Laboratory or the Field

This broad-spectrum acute toxicity bioassay is available in both a lab version (Microtox) and a portable field version (Deltatox). Both versions are based on a bacterial bioluminescence test described in method 8050 of *Standard Methods* (1998). The assay is a metabolic inhibition test that uses a suspension of the luminescent bacteria *Vibrio fischeri* as the test organism. The assay involves adding approximately 10^6 of these bacteria (stored in a freeze-dried condition) into a water sample and measuring the resulting light output of the test bacteria with a photometer after 5, 15, or 30 min of exposure.

If an acutely toxic substance is present in the sample, the substance can negatively affect the cellular structure or metabolism of the bacteria, resulting in a measurable decrease in the intensity of bioluminescence. By comparing the change with the luminescence measured on a negative control sample (i.e., bacteria suspended in reagent-grade water), the system's software can indicate the presence of a toxic substance and gauge the intensity of the toxin's effect, expressed as a percentage reduction in light emitted.

Eclox System Is a Simple-to-use Water Assay

Developed as a field-testing kit for the British military, Eclox was originally intended to enable soldiers to perform rapid tests first on raw water to determine treatability and subsequently on treated water to help confirm safety for consumption. However, the simplicity and speed of testing also make the kit attractive for emergency use by water utilities investigating the threat of intentional contamination.

The Eclox screen is a rapid chemiluminescence assay for the presence of toxic substances. The test uses a plant enzyme that produces light when mixed with other reagents. When present in water, certain pollutants interfere with the chemical reaction and reduce the amount of luminescence. The extent of inhibition of chemiluminescence is assumed to be proportional to the concentration of the contaminant.

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* Aqua Survey Inc., Flemington, N.J.

Specifically, a distilled water blank and subsequently water samples are added individually to a mixture of the chemical luminol, a reaction enhancer (para-iodo-phenol), an oxidant, and the plant enzyme horseradish peroxidase. The biochemical oxidation–reduction reaction releases free radicals, resulting in stable chemiluminescence. The light released is measured over a 4-min period. Any substance in the water sample that absorbs free radicals (e.g., urine or fecal material) or attacks the enzyme (e.g., phenols, amines, or heavy metals) will reduce light output. The light released when bacteria are exposed to the sample water is compared with that from bacteria exposed to a distilled water reference. The extent (percentage) of inhibition, indicated by reduction in luminescence, is a measure of relative water quality.

IQ Toxicity Test Is Based on a Different Test Mechanism

Another acute toxicity screening method that is commercially available is the IQ Toxicity Test. Rather than use a chemical reaction or bacteria, this assay relies on an aquatic invertebrate as a toxin indicator. The freshwater crustacean *Daphnia magna* (water flea) is small but visible to the naked eye. It has been used for years as an indicator organism in both acute and chronic toxicity assays.

In this acute assay, the toxicity of a water sample is characterized by observing stressor-related suppression of *Daphnia* enzyme activity. A group of six daphnids is placed into each of a series of clear 10-mL exposure chambers, some containing negative control water (reagent-grade water) and some containing the water to be tested. After the organisms are in contact with the control and sample waters for a 1-hr period, a fluorogenically tagged sugar suspension is added to each of the compartments. After an additional 15 min, the exposure chambers are illuminated with long-wave ultraviolet radiation (black light). The healthy control organisms should have ingested the tagged sugar (galactose) and expressed the enzyme (galactosidase). The marker, although unable to fluoresce when attached to the sugar molecule, is now liberated and fluoresces as it flows through the organism's circulatory system. If the daphnids suspended in the sample water are not glowing like those in the control chamber, they are considered to be adversely affected by toxic substances in the water sample.

Microtox and Eclox Evaluated for Effectiveness

An effective broad-spectrum rapid screening method should demonstrate the following characteristics:

- The test is capable of detecting multiple toxins (including contaminants of concern).
- The test can achieve detection levels lower than those of public health concern.
- The test is quantitative.
- The test is reproducible.
- False-positive and false-negative results are minimal.
- Performance of the test requires minimal time.
- Performance of the test requires minimal analytical skill.
- Test equipment and supplies are available at minimal cost.

To evaluate the effectiveness of Eclox and Microtox, the following sets of analyses were performed:

1. To determine sensitivity to various toxins, side-by-side analyses were conducted using Microtox and Eclox on distilled water spiked with various concentrations of the following substances: phenol, cyanide, a commercial rodenticide,[*] and a commercial pesticide.[†]
2. To assess the influence of potential interferences from drinking water treatment chemicals, analyses were conducted on distilled water containing various concentrations of copper.
3. To assess matrix effects on the screening analyses, a series of tests were run on distilled water, river water, groundwater, and finished drinking water spiked with various cyanide concentrations.
4. To establish the normal background luminescence levels for these tests, river samples and finished water samples (from the PWSA drinking water treatment plant and five sites in the distribution system) were routinely screened by both methods over a period of months.

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[*] d-Con, Reckitt & Colman Inc., Wayne, N.J.

[†] Bug-B-Gon, Monsanto Inc., San Ramon, Calif.

5. Microtox and Eclox screenings were performed on water samples collected from the routine distribution sampling sites (described previously) during periods of heightened national security awareness (e.g., on elevation of US national alert level from yellow to orange). Daily screening of water from several of these sites is one of the additional security precautions taken by PWSA during periods of heightened alert.

Results Point Up Method Strengths and Weaknesses

Sensitivity

Figures 1 and 2 show Eclox and Microtox results for distilled water spiked with various concentrations of the commercial rodenticide. Results showed significant variation ("noise") among repeat samples for both Eclox and Microtox. In this evaluation, a percentage inhibition of 20% was chosen to indicate the detection limit of the screening test to a potential toxin. Table 1 summarizes contaminant detection limits for cyanide, phenol, the rodenticide, and the pesticide using this criterion. As shown in the table, apparent detection limits for Eclox were 0.05 mg/L for phenol and 0.01 mg/L for cyanide. Apparent detection limits for Microtox were 1.0 mg/L for phenol and 0.2 mg/L for cyanide. Yet both screening methods appeared to detect approximately the same low concentrations of the rodenticide (0.5 mg/L) but only much higher concentrations of the pesticide (10 mg/L).

Interferences

Figures 3 and 4 show potential interferences from plant treatment chemicals on the detection of toxins. Microtox readily detected copper whereas Eclox did not. This sensitivity to copper has the potential to act as an analytical interference at those municipal systems that rely on copper sulfate for algae control.

Matrix effects

It was important to determine the extent to which results of the rapid screening techniques were influenced by the type of water being analyzed. As Figures 5–7 indicate, Microtox and Eclox readings for samples spiked with various concentrations of cyanide were similar for river, tap, and well waters. Compared with other water types, however, distilled water spiked with cyanide tended to yield relatively higher values in the Microtox test (Figures 6 and 7) and lower values in the Eclox test

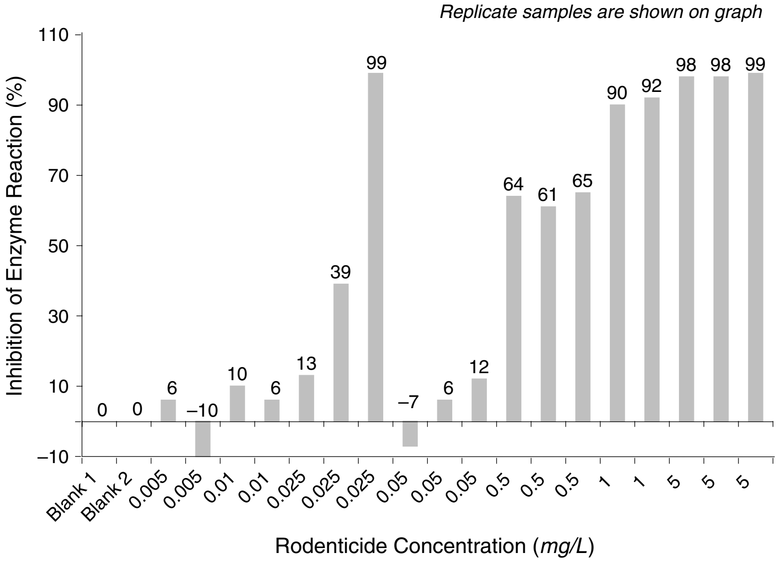

Figure 1 Eclox assay of distilled water spiked with rodenticide

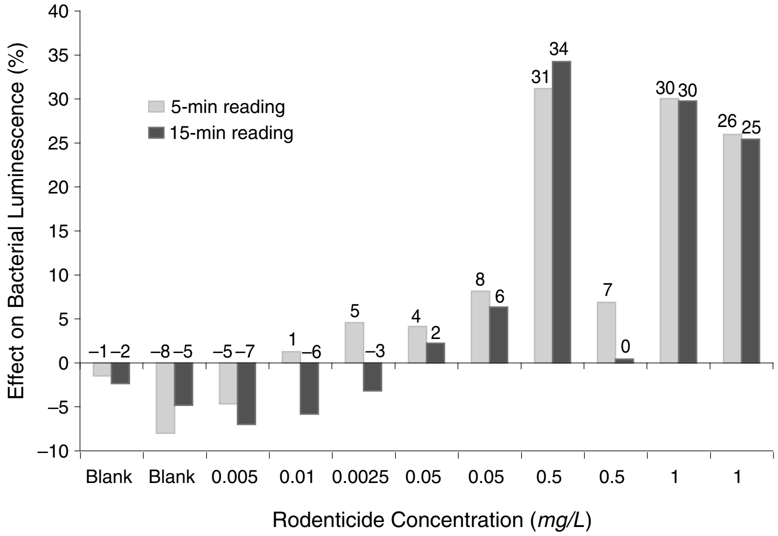

Figure 2 Microtox assay of distilled water spiked with rodenticide

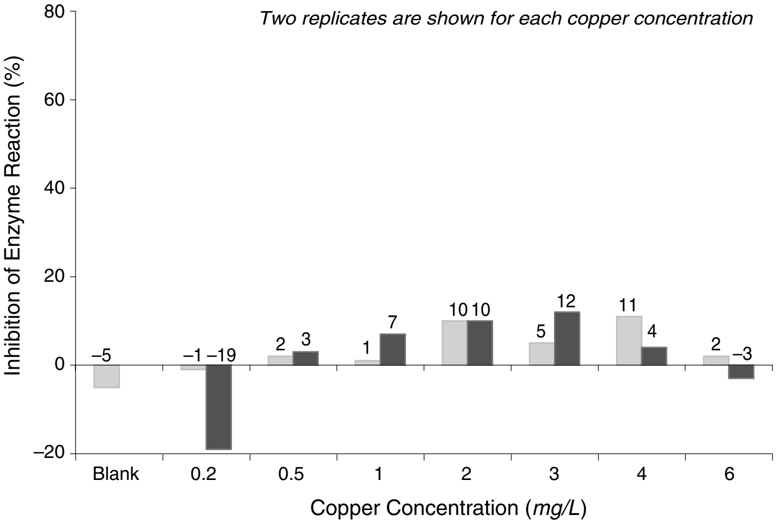

Figure 3 Eclox assay of distilled water spiked with copper

Table 1 Eclox* and Microtox† contaminant detection limits

Contaminant	Eclox mg/L	Microtox mg/L‡
Cyanide	0.01	0.2
Phenol	0.05	1.0
Rodenticide§	0.5	0.5
Pesticide**	10.0	10.0

* Severn Trent Services, Oxfordshire, England

† Strategic Diagnostics Inc., Newark, Del.

‡ Combined results of 5- and 15-minute Microtox readings

§ d-Con, Reckitt & Colman Inc., Wayne, N.J.

** Bug-B-Gon, Monsanto Inc., San Ramon, Calif.

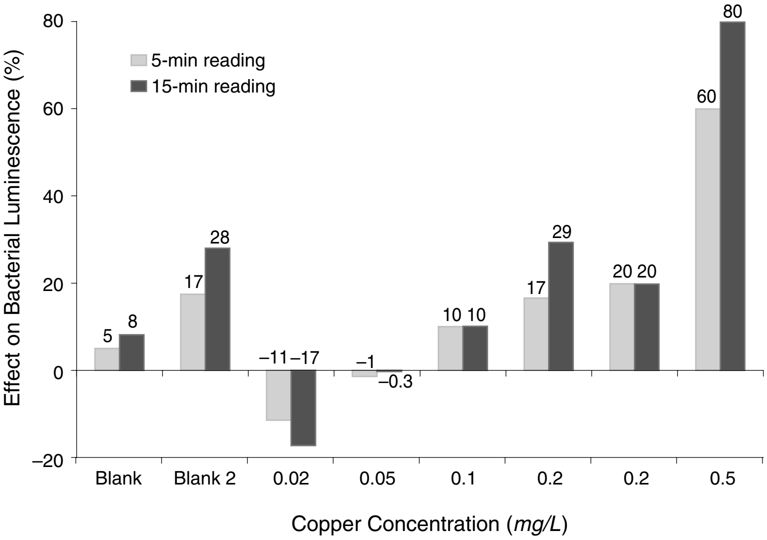

Figure 4 Microtox assay of distilled water spiked with copper

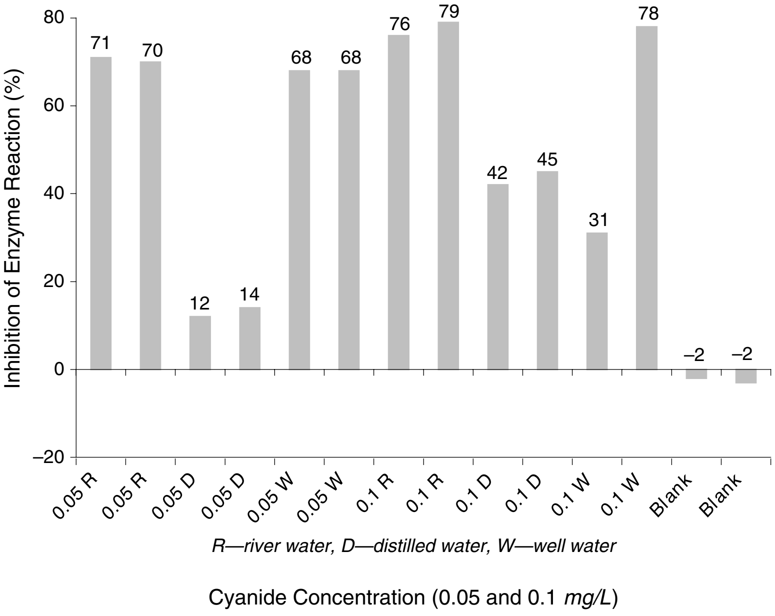

Figure 5 Eclox matrix effects of cyanide in three types of water

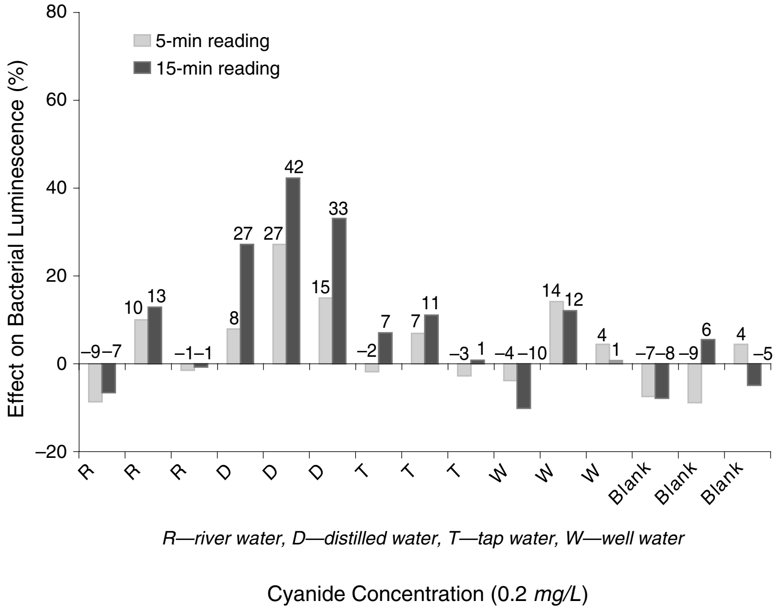

Figure 6 Microtox matrix effects of 0.2 mg/L of cyanide in four types of water

(Figure 5). These matrix effects were also observed when cyanide was spiked into tap water at various concentrations (data not shown).

Background Samples Were Taken From Various System Locations

In an effort to assess the significance of an individual result from one of the rapid screening tests, it is first necessary to establish baseline values on waters collected from various portions of the treatment and distribution system. It is also important to establish predictable changes in readings that might be associated with such factors as changes in season or treatment conditions.

Samples of river water were collected from the PWSA treatment plant intake as well as finished water supplies from the treatment plant and five representative sites throughout the distribution system. The samples were collected at various times during a one-year period and analyzed by Microtox and Eclox to obtain baseline data.

Table 2 shows the baseline Eclox data for these samples. The mean values for all distribution system sample sites were quite low and similar to those for the distilled water blank. Mean values for the river and

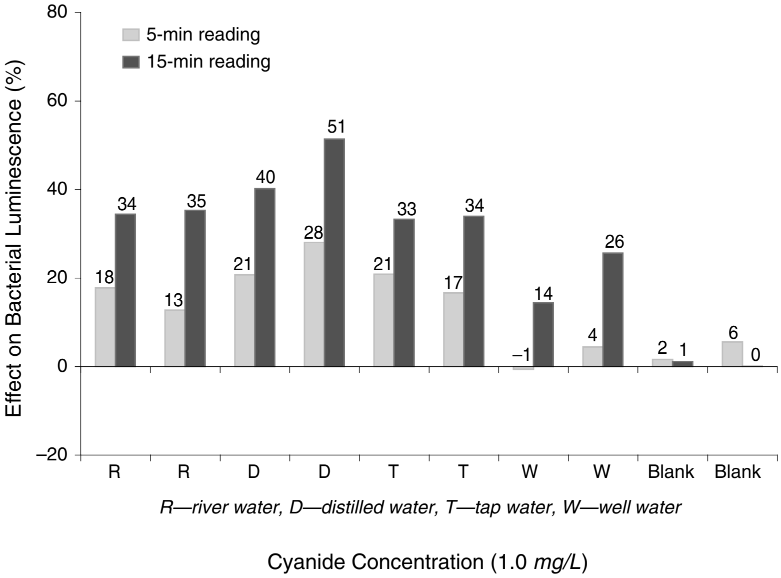

Figure 7 Microtox matrix effects of 1.0 mg/L of cyanide in four types of water

treatment plant finished water were much higher. Typical river water readings had a mean of 56. Plant finished water readings were somewhat lower than those for the river water, with a mean of 32. The reason for these differences is not known.

Table 3 summarizes the results for the 15-min Microtox tests. The mean values for all sample sites remained close to zero. However, as indicated by the maximum values, minimum values, and standard deviations, individual readings varied significantly.

Some Samples Were Collected During Periods of Heightened Security

One of the precautionary security measures taken by PWSA during periods of heightened alert is daily acute toxicity screening of samples from the river and several of the distribution sites routinely analyzed by Eclox and Microtox. The purpose of this sampling is to detect changes from the norm that could signal the presence of toxic substances.

Figures 8 and 9 show precautionary screening results for distribution site 3 during a period in March–April 2003 when the national alert level was elevated from yellow to orange. The Microtox and Eclox routine screening results for this period were relatively low. As the figures

Table 2 Total Eclox[*] routine samples for 2003

Sample Location[†]	Maximum[‡]	Minimum[‡]	Mean[‡]	Standard Deviation[‡]	Number of Samples
Blank	26	–18	1	13	17
Raw (river water)	75	–21	56	27	17
Finished water	65	–28	32	18	20
Site 1	31	–30	5	14	25
Site 2	38	–12	3	10	25
Site 3	28	–19	3	11	25
Site 4	25	–30	6	12	24
Site 5	15	–21	2	9	25

[*] Severn Trent Services, Oxfordshire, England

[†] Sites 1–5 designate distribution system sample sites. The raw and finished water samples were taken at the Pittsburgh (Pa.) Water and Sewer Authority Water Treatment Plant.

[‡] Percent inhibition of enzyme reaction

Table 3 Total 15-min Microtox[*] routine samples for 2003

Sample Location[†]	Maximum[‡]	Minimum[‡]	Mean[‡]	Standard Deviation[‡]	Number of Samples
Blank	14.0	–11.3	–2.1	5.4	27
Raw (river water)	27.9	–35.6	–6.9	10.4	29
Finished water	13.5	–24.2	–4.9	8.9	24
Site 1	30.5	–24.1	–6.1	11.5	26
Site 2	20.5	–49.2	–6.6	13.6	26
Site 3	12.7	–45.5	–8.5	10.5	27
Site 4	35.2	–24.8	–4.5	11.8	24
Site 5	23.9	–30.3	–6.3	11.3	25

[*] Strategic Diagnostics Inc., Newark, Del.

[†] Sites 1–5 designate distribution system sample sites. The raw and finished water samples were taken at the Pittsburgh (Pa.) Water and Sewer Authority Water Treatment Plant.

[‡] Percent impact on bacterial luminescence

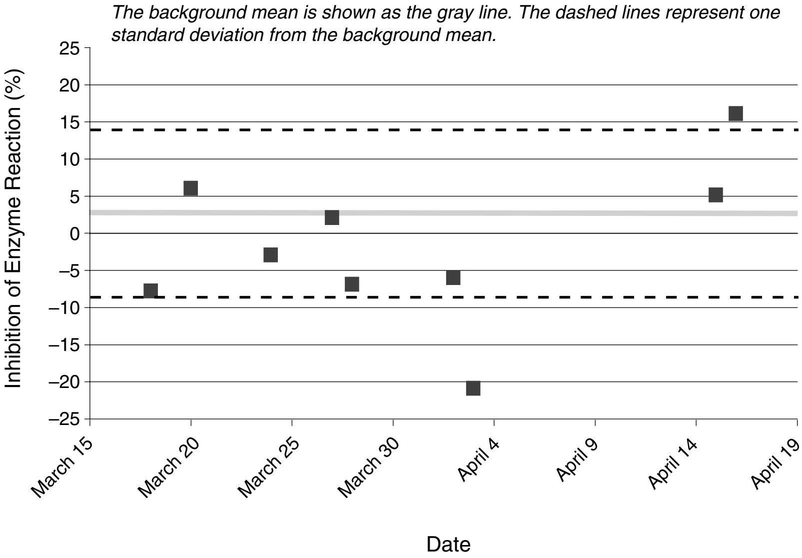

Figure 8 Orange alert Eclox data for sample site 3

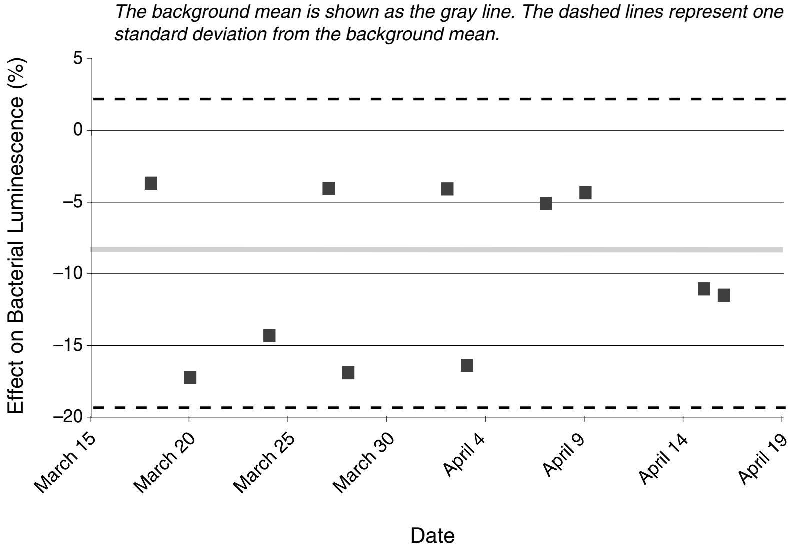

Figure 9 Orange alert 15-min Microtox data for sample site 3

indicate, they generally remained below the arbitrarily chosen control limit of one standard deviation from the mean values established during routine sampling. In accordance with the standard operating procedure in the PWSA laboratory, a sample result exceeding the one standard deviation control limit would indicate the need for a repeat analysis of the sample. Should the repeat screening result also exceed the control limit, resampling and perhaps other types of analyses would be required.

SUMMARY AND CONCLUSIONS

To comply with USEPA guidance on response to suspected or actual incidents of intentional contamination of water supplies, utility and nonutility responders need rapid assays that can be performed in the field. These assays must be able to screen for biological, chemical, and radiological substances that may pose an acute hazard to individuals investigating a site of suspected contamination. They should also presumptively help identify or rule out contaminants that might be present in the water.

This research surveyed commercially available analytical techniques that might be used by response teams from water utilities or other organizations for site characterization and evaluated the techniques' strengths and limitations. The assays discussed represented test procedures for a variety of potential contaminants, including certain chemical and biological substances considered to be credible agents for attacks on water supplies.

Technologies Summarized

Table 4 summarizes the technologies reviewed in this chapter. In addition, the following sections offer a brief assessment of each of the assays evaluated in the study.

Rapid immunoassays

These tests may be suitable for screening for acute hazards that pose a threat to the site characterization team. However, poor sensitivity limits their usefulness in accurately detecting low levels of contaminants in water samples. Additional work is needed on field concentration techniques to improve sensitivity and on potential cross-reactivity of these assays with nontarget agents.

Table 4 Summary of rapid analytical techniques evaluated

Assay or Test	Product Name	Manufacturer	Contaminants Detected	Detection Limits*	Difficulty in Performing Analysis	Time† *min.*
Rapid immunoassays	BTA test strips	Tetracore	Pathogens (anthrax, plague, tularemia, cholera)	Pathogenic bacteria: 10^5 cfu/mL	Simple	15
	SMART tickets	New Horizons Diagnostic	Biotoxins (*Clostridium botulinum*, ricin, staphylococcal enterotoxin B)	Biotoxins: 2–50 µg/L (ppb)		
Rapid enzyme test	Pesticide/ nerve agent	Severn Trent Services	Insecticides (organophosphates, carbamates, thiophosphates)	Insecticides: 0.1–5 mg/L (ppm)	Simple	5
			Nerve agents	Nerve agents: data not available		
PCR‡	R.A.P.I.D.	Idaho Technology	Pathogens (anthrax, plague, tularemia, *Brucella*, *Camphylobacter*, *Escherichia coli* 0157, *Salmonella*, *Listeria*)	$\sim 10^3$ cfu/mL	Moderately difficult	90
			Biotoxins (*C. botulinum* indirectly)			

Table continued next page.

Table 4 Summary of rapid analytical techniques evaluated *(continued)*

Assay or Test	Product Name	Manufacturer	Contaminants Detected	Detection Limits[*]	Difficulty in Performing Analysis	Time[†] *min.*
Field-deployable GC/MS[§]	HAPSITE	INFICON Inc.	Volatile organic compounds	Low µg/L–mg/L	Most difficult	60
Acute toxicity screening methods	Eclox	Severn Trent Services	Industrial chemicals, weaponized chemicals, biotoxins	µg/L–mg/L	Simple	5
	Microtox	Strategic Diagnostics Inc.			Moderately difficult	45
	IQ Toxicity	Aqua Survey Inc.			Moderately difficult	90

[*] Manufacturer's reported detection limits without sample concentration

[†] Run time is for the field test (including sample preparation). Most of these methods allow more than one sample to be run simultaneously.

[‡] PCR—polymerase chain reaction

[§] GC/MS—gas chromatography/mass spectrometry

Rapid enzyme test

This assay is a rapid screen for nerve agents and certain pesticides and is easily conducted using either concentrated or nonconcentrated sample water.

Rapid PCR

As was the case for commercially available rapid immunoassays, the effectiveness of this test is limited by its sensitivity. A field concentration technique with adequate recovery is needed to make this assay useful for more than just screening for acutely hazardous conditions at the site being investigated. One approach for further increasing the sensitivity of both rapid immunoassay and rapid PCR involves field concentration by filtration, possibly followed by further laboratory concentration via centrifugation of the field concentrate.

Field-deployable GC/MS

This system offers rapid identification of volatile trace organic compounds at low levels in the field. The newly added in situ purge-and-trap system facilitates sample collection from air and water and enhances sensitivity.

Acute toxicity screening

Both Microtox and Eclox detect similar concentrations of a variety of toxins. Detection limits for some contaminants appear to be substantially lower than for others. However, neither assay provides consistently reproducible results. In this study, variations were observed in replicate sample analyses, and these suggest that limited confidence can be placed on any single sample result.

For both Microtox and Eclox, responses to individual contaminants may not be linear. Plant treatment chemicals (e.g., copper) may interfere with the assays, and these potential interferences must be considered when Microtox and Eclox results are interpreted. Furthermore, actual values generated by these screening techniques may vary in different types of water, especially distilled water. In order to assess the significance of an individual screening result from a water system sample, it is necessary to first establish the baseline value for that site. Screening results obtained during an emergency or during a period of heightened security can be interpreted by plotting the values on a control table constructed using background screening data.

The authors believe that results from acute toxicity screening are most appropriately interpreted as follows. Negative results do not guarantee the safety of the water. Positive results suggest the possible presence of a toxic substance and indicate the need for more specific analyses.

Further Research and Testing Are Needed

Although the capability and efficacy of rapid, on-site analytical techniques are continually improving, the results generated by these field-screening technologies are considered presumptive and are not a substitute for standard laboratory analyses. Rather, these assays are intended to provide information for rapid hazard assessment and timely selection of operational and public health responses. Results generated by these analyses must be interpreted with caution.

Additional studies are required to assess matrix effects and the influence of interferences on these assays. For some of the assays, testing is needed to establish the incidence of false-positive and false-negative results. Continued evaluation is necessary to establish whether the techniques' detection limits (with or without sample concentration) are low enough to determine the presence of contaminants at levels of public health concern.

ACKNOWLEDGMENTS

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CHAPTER · 29

Critical Review of Emerging Technologies for Rapid Detection of Drinking Water Contaminants

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BACKGROUND

Parallel with new security-related detection needs for water utilities, research in rapid detection techniques for both chemical and biological contaminants is on the verge of producing easy, fast detection of many contaminants. This chapter identifies contaminants that may be used in intentional releases into potable water systems, reviews currently available techniques for detecting those contaminants, explores emerging detection technologies, and recommends a course of action for water utilities developing a monitoring program. The contaminants identified in the literature as potential threats to potable water are diverse and include spore-forming and non-sporulating bacteria, DNA and RNA viruses, protozoa and biological toxins, chemical warfare agents, and organic and inorganic chemicals. The general features by which contaminants may be judged as potential threats are presented. Among detectors of replicating biological agents, polymerase chain reaction (PCR)–based methods stand out as the only currently available technology capable of detecting the diversity of agents identified as potential contaminants at the sensitivity and in the time frame required. In the absence of a single chemical detector for detection of all potential contaminants, it is recommended that utilities purchase equipment such as chemiluminescence or bioluminescence test kits for field testing in the event of a suspected release and develop pilot monitoring programs to pave the way for implementation of improved chemical monitors as they are developed.

INTRODUCTION AND OBJECTIVES

The work reported herein is presented to help utilities select appropriate devices and strategies for monitoring water quality and maintaining security in water distribution systems. Information in this chapter was collected as part of a project that Drexel University performed for the Philadelphia Water Department. The project entailed identifying priority biological and chemical contaminants, surveying available detectors and those that may be available in the near future, and assessing the practicality of including available detectors in a distribution system monitoring regimen.

This report begins with a discussion of how contaminants might be introduced into a drinking water system and what contaminants might be introduced. Following the discussion of threats, the devices commercially available for detecting those agents are enumerated. A general discussion of detection of waterborne contaminants follows. Finally, the best detectors currently available for biological and chemical agents are recommended along with strategies for implementing them in a monitoring program.

Numerous commercial devices are named in this chapter. Data used in assessing these products were collected via the scientific literature, discussions with manufacturer representatives, and discussions with researchers who use those devices. Omission of products from this discussion should be considered an oversight of the authors and not reflect negatively on those devices.

THREATS AND AGENTS

Characteristics of Intentional Releases

We conceive of two sorts of contaminant releases into a drinking water distribution system: a “sophisticated” release and an opportunistic release.

Someone knowledgeable of drinking water distribution system operations and toxicity would perpetrate the sophisticated release. The perpetrator would have access to chemicals, biological materials, or radioactive materials whose purchase and use are restricted and would be capable of handling the materials without harm to himself or herself. This person would be able to select and access points in the distribution system at which he or she could make the release effectively and without detection. The sophisticated release has the potential for causing harm to a large number of customers and panic among customers not directly affected. Those less knowledgeable will make opportunistic releases. They will use readily available materials and will introduce them into the drinking water system in easy-to-access locations. Opportunistic releases will likely involve chemical agents, since biological and radiological agents require greater skill in handling. Releases will produce relatively little harm to customers, but pose the potential for degrading customer confidence or exposing vulnerabilities.

Historically, attempted attacks on water systems have fallen between these extremes, as with the threats or attempts at poisoning public water supplies perpetrated by white supremacists, environmental terrorists, and international terror organizations over the last several decades (Beering, 2002). Like Luthy (2002), we believe top priority in developing responses to threats to water security should be in protecting water storage and transmission structures, and like Rose (2002), we believe monitoring and detection should play a pivotal role. Though source water protection is surely merited as a part of enhanced water system security, dilution and treatment will mitigate the impact of releases into reservoirs or source waters. In distribution systems, dilution is less pronounced than in raw water supplies, and in current practice monitoring and detection are sparse if existent at all.

Review of Literature Pertaining to Drinking Water Contamination

Biological contaminants

Biological contaminants may be biological warfare agents or other living organisms or toxins. Biological warfare agents may have been weaponized (have modified transport properties or infectivity) and may have been produced in large quantities by nations pursuing weapons programs or terrorist groups.

Burrows and Renner (1999) assessed the viability of biological warfare agents as threats to potable water. Eighteen agents were selected from among potential biological agents based on the likelihood that they have been weaponized or are present in the arsenal of a nation or group. Ingestion data were not available for some agents and their viability as potable water contaminants was based on inhalation or aerosol exposure toxicity data. Living biological contaminants were assessed as likely, probable, or possible threats to potable water based on their infectivity, stability in water, and tolerance to chlorine. Toxins were assessed according to their NOAEL (highest no-adverse-effect level based on consumption expected by adult military personnel), stability in water, and tolerance to chlorine. In summarizing their survey, Burrows and

Renner note that, although most weaponized agents have been developed for aerosol dissemination, many pose a credible drinking water threat. Numerous bacteria were classified as likely or probable threats to potable water. Among Rickettsia-borne diseases, they identify only Q fever as a credible threat. Among viruses, enteric viruses and possibly variola are presented as credible threats. All the toxins studied are considered viable waterborne threats, though the authors believe only botulinum toxins, ricin, Staphylococcal enterotoxin B (SEB), trichcothecene mycotoxin (T-2 mycotoxin), and possibly aflatoxin are available in sufficient quantity to pose credible threats. Diseases classified as unlikely due to waterborne releases were glanders, melioidosis, typhus, encephalomyelitis, and hemorrhagic fever.

Hickman (1999), in his assessment of biological and chemical risks to U.S. Air Force water systems, mentions bacteria and toxins as agents best suited for terrorist attacks. *Shigella* spp. and *Vibrio cholera* are mentioned as solely waterborne threats (as opposed to agents that were selected or developed as inhalation threats). Other definite or probable biological water threats Hickman provides are *Bacillus anthracis*, *Brucella* spp., *Clostridium perfringens*, *Francisella tularensis*, *Shigella* spp., *Vibrio cholerae*, *Salmonella typhi*, *Yersinia pestis*, *Coxiella brunetti*, *Variola major*, Hepatitis A virus, and *Cryptosporidium parvum*.

Like Khan et al. (2001), we consider "naturally occurring" outbreaks to be an indicator of vulnerability and a source for data on potential contaminants. Khan et al. note that "mundane" agents such as *Salmonella* and cyanide have already been used in terrorist attacks on food and water and should not be overlooked. Khan et al. state that dilution, disinfectant residual, environmental stability of biological agents, and the relatively small amount of tap water directly consumed make waterborne terrorism less likely than other routes. Nonetheless, they indicate that *Cryptosporidium* poses a specific threat and that data are not yet available on toxicity of oral exposure to many biological warfare agents or the resistance of many agents to chloramines. Among illness outbreaks from drinking water systems between 1991 and 1998, Craun et al. (2002) state that 41% were of bacterial, viral, or protozoan etiology; 18% were due to chemical causes; and the remainder were due to unknown causes. Biological agents responsible for outbreaks during that time period included *Giardia*, *Cryptosporidium parvum*, Norwalk-like viruses, *Campylobacter*, non-typhoid *Salmonella*, *E. coli*, *Shigella*, *V. cholerae*, Hepatitis A virus, and *Plesiomonas shigelloides*. Among outbreaks not related to source water contamination, more than half were due to distribution system contamination, the route by which terrorists might produce the most effective attack.

Members of an expert panel assembled to discuss potential contaminants and detection at the 2002 AWWA Water Technology Conference (Seattle, Wash., November 2002) were asked to identify which contaminants they considered the most dangerous. In general, panel members agreed that biological agents posed the greatest threat due to their ability to cause harm at a very low dose, the relative ease with which they may be obtained or created, and the difficulty in detecting them. Anthrax was considered the most problematic agent because of its potency and resistance to disinfection. In addition to biological agents, panel members pointed to the hazard posed by commonplace chemicals such as pesticides. One panelist thought LSD was the most dangerous agent.

Deininger (2000) assessed contaminants selected by the Federal Emergency Management Agency (FEMA) in regard to their threat to public water systems. He concluded that, in general, biological toxins and bacteria pose the greatest threat, followed by some viruses, then protozoa. Nerve agents, blood and blister agents, and toxic chemicals were considered less likely agents. Deininger mentions botulinum toxins as the most serious biological toxin threat, though he points out that botulinum toxin is readily deactivated by chlorine. Although he regards viruses as a relatively low threat, Deininger comments that the resistance of viruses to disinfectants and the infectious doses are not yet well characterized. Deininger does not consider *Cryptosporidium parvum* a credible threat to water supply systems because its effects are relatively minor. Finally, among living biological agents, Deininger considers *Bacillus anthracis*, *Shigella*, *Salmonella*, and *Vibrio cholerae* real threats. Deininger points out that only *B. anthracis* is unaffected by normal disinfectant concentrations present in distribution systems.

In reviewing a list of biological priority agents prepared by the authors of this chapter, the Philadelphia Water Department (2003) indicated that airborne biological threats, particularly *Legionella pneumophilia*, pose a drinking water challenge and should be considered among priority agents. *Legionella* has caused numerous inhalation-related outbreaks since its association with an outbreak in Philadelphia in 1976.

Finally, in the most comprehensive study of potential biological contaminants to drinking water systems identified, Teter (2002) performed a threat analysis for many biological and chemical agents based on their relative health effects, environmental fate, and ease of attainment and use. In general, Teter determined that biological agents pose a much greater threat than chemical agents, with *Bacillus anthracis*, *Baylisascaris* *procyonis*, *Clostridium perfringens*, and *Cryptosporidium parvum* rated the most important agents due to their chlorine resistance.

Table 1 summarizes the biological agents identified as potential drinking water threats in the literature. Though this list is incomplete, it presents some of the most problematic agents and demonstrates the diversity of biological agents utilities must contend with in developing monitoring strategies. The literature review of biological contaminants revealed the need for research on the stability of potential biological contaminants in water, their resistance to treatment, and particularly their tolerance to disinfection residuals in the concentrations present in distribution systems. The properties of these agents may vary significantly from strain to strain or between weaponized and non-weaponized versions.

Chemical contaminants

General categories of chemical agents identified in the literature as threats for drinking water contamination are: chemical warfare agents; toxic industrial chemicals (with explicit mention of cyanides [sodium cyanide in particular], arsenic-containing compounds, fluoride-containing compounds, solvents, and explosives); pesticides, insecticides, and rodenticides; and metals. The literature suggests that chemical agents pose less threat for drinking water system contamination than biological agents and that factors safeguarding water systems from deliberate contamination are dilution, action of disinfectant residual, and the relatively small amount of water ingested directly from tap water (Khan et al., 2001). Although experts at the AWWA Water Quality Technology Conference panel discussion on biological and chemical agent detection (Seattle, Wash., November 2002) and the USEPA Workshop on Detector Technology (Newark, N.J., December 2002) acknowledged the importance of readily available substances as threats to water supply, the literature surveyed generally focused on exotic, toxic substances—most often chemical warfare agents. Examples of chemical contaminants that have been accidentally introduced into source waters are (Grayman et al., 2001): phenol, aniline, carbon tetrachloride, fuel oil, toluene, ethylene dibromide, petrochemicals, pesticides, formaldehyde, free cyanide, and ammonia.

Several authors have published lists of chemicals or groups of chemicals that may be used to contaminate potable water systems. Hickman (1999) states that the chemical warfare agents hydrogen cyanide, Tabun, Sarin, Soman, VX, Lewisite, sulfur mustard, BZ, and LSD are threats to military potable water systems. Other agents (non-chemical warfare agents) Hickman considers a concern are organophosphate

Table 1 Summary of biological contaminants mentioned in the literature

Type of Contaminant	Contaminants Mentioned in the Literature as Potential Threats
Spore-forming bacteria	*Bacillus anthracis*, *Clostridium perfringens*
Nonsporulating bacteria	*Brucella* spp., *Campylobacter*, *Corynebacterium diptheriae*, *E. coli* O157:H7, *Francisella tularensis*, *Legionella pneumophilia*, *Neissera meningitides*, *Salmonella typhi*, *Shigella* spp., *Variola major*, *Vibrio cholerae*, *Yersinia pestis*
DNA viruses	*Variola major*
RNA viruses	Alphaviruses, Arenaviruses, Bunyaviruses, Filoviruses, Flavivurses, Hepatitis A, Norwalk-type viruses
Protozoa	*Cryptosporidium parvum*, *Giardia*
Rickettsia	*Coxiella brunetti*
Toxins	Abrin, Aflatoxin, Anatoxin A, Botulinum toxin, *Clostridium perfringens* epsilon toxin, Conotoxins, Diacetoxyscirpenol, Microcystins, Ricin, Saxitoxins, Shigatoxin, Staphyloccocal entrotoxins, T-2 Micotoxins, Tetrodotoxin

pesticides (Malathion, methyl parathion, and chlorpyrifos), organochlorine pesticides (lindane, dieldrin, methoxychlor, and endosulfan), rodenticides (sodium fluoroacetate, strychnine, and thallium sulfate), and some inorganic chemicals (sodium cyanide, arsenic, fluoride, and mercury). Khan et al. (2001) broadly identified potential chemical contaminants as disinfection by-products, pesticides and herbicides, inorganic chemicals and radionuclides, and arsenic-containing compounds. Johnson et al. (1999) reviewed the exposure guidelines for military personnel for an extensive list of toxic chemicals. The chemicals were grouped according to their toxicity or whether they were developed as chemical warfare agents. “High-priority” chemicals included pesticides and herbicides and some toxic industrial chemicals.

Deininger (2000) does not consider nerve agents a credible threat to water supplies. The factors he lists as important in determining whether a toxic chemical is a threat are its solubility and its volatility. He states that highly volatile chemicals do not pose a serious threat, since vapors would be easily detected and water would not be consumed.

Other agents that should be considered when developing a list of priority chemical contaminants can be drawn from the USEPA's Drinking Water Standards and Health Advisories (USEPA, 2002) and the Agency for Toxic Substance and Disease Registry Minimal Risk Level database (ATSDR, 2003). Chemicals with a very low effective dose for oral exposure should be considered.

Characteristics of Credible Threats

Although various organizations are developing lists of priority contaminants, it is advisable that all utilities develop their own lists. In contrast to generic lists, these lists will account for the type of treatment employed at the utility, the availability of particular agents in the vicinity of the utility, and the analytic capabilities of the utility.

The following characteristics should be considered when developing threat lists:

- Toxicity (or effective dose)
- Solubility (for chemical agents)
- Volatility (for chemical agents)
- Availability
- Fate in treatment
- Stability
- Disinfectant residual tolerance
- Color, taste, and odor

Among these characteristics, toxicity (or infectivity) and solubility may be the most important, since these characteristics are most directly related to the exposure and hazard of the agent.

DETECTION AND MONITORING

Viable and Available Detectors

Viable, commercially available detection technologies for biological and chemical agents are summarized in Table 2. Detectors were chosen based on several use scenarios and correspond to the contaminants described above. To be considered for evaluation, instruments or techniques must

Table 2 Summary of available viable detector types

Detecting Biological Agents	Detecting Chemical Agents
Broad-spectrum methods	*Broad-spectrum methods*
ATP/Chemiluminescence	Cross-reactive chemical sensor arrays
Particle counters	Other sensor arrays
	Chemiluminescence
	Toximeters/biomonitors
	UV absorption
	Particle counters
Agent-specific methods	*Agent-specific methods*
Immunoassay	Biosensors
Biosensors	Ion-selective electrodes
PCR	Gas chromatography
Flow cytometry	Liquid chromatography
Microbiometry	Ion mobility spectroscopy
	Inductively coupled plasma spectroscopy

produce results within eight hours and detect agents or surrogates at levels of possible health significance. Eight hours was chosen so that detection could be done within a single shift. Levels of health significance are generally thought of as acute NOAEL (no observed adverse effect level), though for consumer confidence it may be desirable to detect agents at lower levels.

Detectors may be used in one of three ways:

1. To monitor water treatment plant discharge or distribution system water quality;
2. For rapid assessment or contaminant tracking after a known or suspected intrusion; or
3. To augment laboratory capacity to identify unknown contaminants or confirm suspected contaminants.

Broad-spectrum methods are those that detect classes of agents or numerous agents based on their toxicity or properties. Agent-specific methods detect a single agent or class of agents. States et al. (2003), in their evaluation of analytic techniques for detection of biological and chemical agents for drinking water security, assert that the broad-spectrum methods are the best methods currently available for water quality screening and suggest their use in conjunction with laboratory equipment that is more selective and sensitive.

All of the detector types listed in Table 2 are available from multiple vendors except "microbiometry," though a number of the methods are not well tested in drinking water treatment analysis or distribution system monitoring. Field test kits for detecting biological agents via chemiluminescence and bioluminescence are available. Among agent-specific methods, the only biological agent detector that is deployed on-line is the microbiometer. Microbiometry is the name Pointsource Technology, LLC (Escondido, Calif.), has given to its one-of-a-kind device for on-line recognition of microorganisms. Among chemical agent detectors, biosensors and liquid chromatography have the potential to be deployed on-line, but have been tested in this mode only in research settings.

In this report, biosensors are sensors that integrate a bioreceptor (organism, antibody, or enzyme) with signal generation equipment (e.g., generation of an electrical signal or fluorescence). An immunoassay operates on the same principle as a biosensor but does not have integrated signal detection equipment. Cross-reactive chemical sensor arrays (a.k.a. electronic noses and tongues) are arrays of sensors that detect water quality parameters and use those parameters to deduce the composition of the medium. Other sensor arrays are arrays whose outputs are water quality parameters or species concentrations (e.g., pH, temperature, conductivity, redox potential [ORP], chlorine, metals concentrations). The difference between cross-reactive chemical sensor arrays and other chemical sensor arrays is that for cross-reactive arrays there is an attempt to deduce the chemistry underlying the sensor signals whereas there is no such attempt for other chemical sensor arrays. Finally, toximeters are devices that monitor the longevity or behavior of living organisms to assess toxicity. In the literature, toximeters have also been called biomonitors or biosensors.

Numerous other detection technologies were identified for this study, some of which are described below. They were not included among the technologies listed in Table 2 because they are not yet available or because their use in detection of agents in drinking water applications is still experimental. Examples of these technologies are PCR-on-a-chip and other lab-on-a-chip biodetectors, infrared optical sensors, and whole cell mass spectrometry.

Research and Trends in Monitoring and Detection

In tandem with the increased need for detection, recent years have witnessed significant improvements in monitoring and detection techniques already in use and a proliferation of experimental techniques that hold promise for water utilities. A comprehensive review of these developments is not possible in this chapter. Rather, highlights are presented below.

Detecting living biological agents

A generalized procedure for detecting biological agents is shown schematically in Figure 1. The boxes shown in solid lines are mandatory for all detection techniques. The boxes shown in dashed lines are used only in select techniques. The two most time-consuming portions of microorganism detection are the culturing or processing of the samples and the actual detection process. “Rapid” techniques are, in general, techniques that speed up one of those two processes (Ventner, 2000).

Harris (2002) quotes biowarfare agent detection experts as saying that the current state of the art in rapid detection involves either antibody-based assays, nucleic-acid-based assays, or mass spectrometry approaches and that the future of biowarfare agent detection lies in refinement of these techniques and miniaturization of detectors. These techniques and others are described below.

PCR. Molecular techniques rely on detection of biological constituents (DNA, RNA, ATP, or potentially other molecules such as lipids) rather than entire organisms. These methods offer the advantage over traditional cell culture methods of faster isolation or production of the target of the detector, though they may produce more false positives than culture-oriented methods due to the presence of target analytes in both viable and non-viable cells (Reynolds et al., 2001). Molecular detection may involve amplification of a cell component (usually DNA or RNA) or direct measurement of an RNA component (since these are far more abundant than DNA) (Morse and Lee, 1997).

PCR, the molecular method in most common use, involves amplification of selected genetic material via (1) conditioning of samples to remove impurities or substances that could confound analysis, (2) release of molecular material from cells, (3) addition of enzymes to cell molecular materials, (4) possibly thermal cycling to promote enzymatic reactions, and finally (5) signal detection. The enzymes are chosen specifically for detecting a given agent and may promote DNA chain

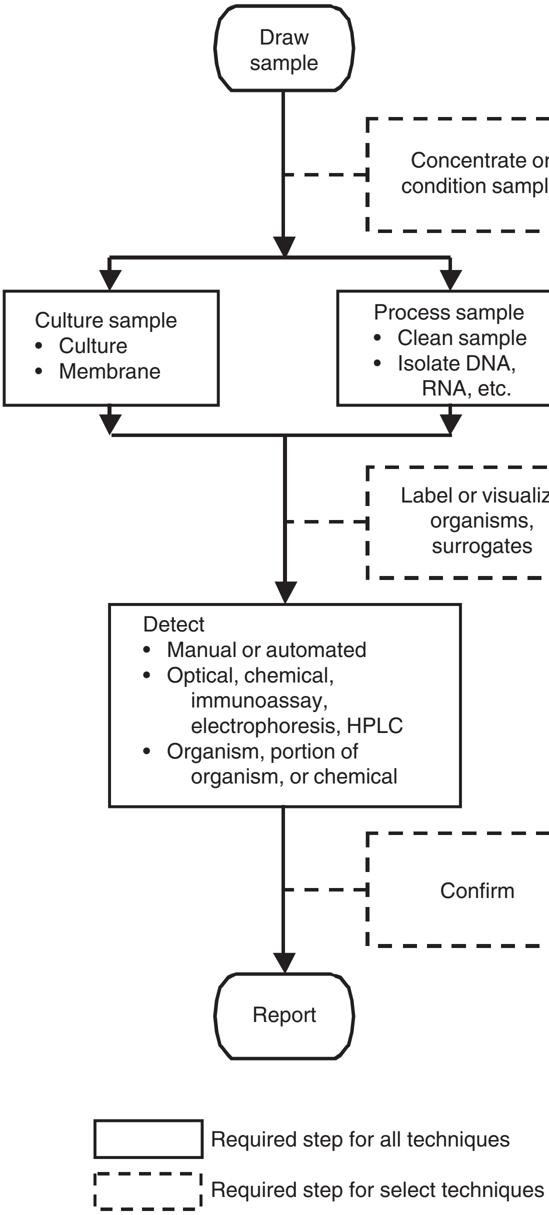

Figure 1 Schematic diagram for detection of microorganisms

growth, transcribe DNA chains into RNA, or transcribe RNA into DNA. Several techniques have been employed for signal detection of amplified genetic material. These include electrophoresis, hybridization, high performance liquid chromatography (HPLC), or ELISA (Ventner, 2000), or more recently sensing the presence of the target product by an acoustic sensor (Deisingh and Thompson, 2001).

Numerous modifications have been made of PCR to allow better amplification or amplification of different molecules (Martin and Timmers, 1997). "Real-time" PCR is the most significant improvement of PCR with respect to its use at water utilities. In real-time PCR the concentration of amplified DNA or RNA is tracked during the amplification process, allowing faster detection and the possibility of quantitative analysis.

PCR techniques are certainly faster than culture-oriented techniques (time on the order of hours versus the order of days), but pose several difficulties (Reynolds et al., 2001):

- Interference by humic substances, metals, and other water components that inhibit the enzymes
- No differentiation between live and dead organisms and nucleic acid in the water matrix but associated with cells
- Specificity of detecting virulent agents depends on primer selection
- Very small sample volume

To overcome some of these difficulties, researchers are commonly employing the integrated cell culture/PCR (ICC/PCR) technique. In this technique a short period of culturing (24 to 48 hours) is performed prior to PCR. Culturing ensures that only viable organisms are present in the sample analyzed.

The future of PCR looks bright, with research ongoing in development of smaller, faster, and better equipment and primers. For example, Waters et al. (1998) report successful development of a microchip device capable of carrying out the basic operations of PCR. The microchip-sized device (overall dimensions are not provided in the report) is submerged in solution containing the biological material of interest and can perform cell lysis, amplification of DNA, and sizing of DNA fragments. Signal detection is performed external to the microchip via fluorescence detection. The authors forecast faster, less labor-intensive PCR procedures as the microchip device is developed further.

OTHER MOLECULAR METHODS. In addition to PCR and other methods that amplify nucleic acid components, some molecular techniques rely on detection of other chemicals associated with pathogenic organisms. For example, spores are "packed full" of dipicolinic acid (DPA) (Casagrande, 2002) and DPA is amenable to detection via electronic noses. Casagrande mentions that electronic noses might detect *Bacillus anthracis* following the liberation of DPA from spores via an enzymatic reaction.

FISH (fluorescence in situ hybridization) probes allow rapid, specific detection of biological agents via capture, labeling, and detecting a portion of the agent's ribosomes (16S rRNA) (Oerther et al., 2002). FISH probes have been used for the detection of many organisms of interest in the water industry, including *Cryptosporidium parvum* oocysts (Vesey, 1998). In that study FISH probes proved feasible for detecting the oocysts, but difficult to use due to weak signals produced in the presence of interfering materials.

Enzymatic methods are available for detection of coliforms, *E. coli*, and potentially other organisms or classes of organisms. These techniques shorten the culturing-detection process via addition of an enzyme to a culture. The enzyme reacts with the culture, yielding a signal strong enough to read relatively early in the process. Commercial offerings of enzymatic methods are currently limited to coliform and *E. coli* detection, but as more enzymatic biosensors (see "Biosensors" under "Detecting chemical agents" below) are developed, it is reasonable to expect enzymatic detection methods to be developed for more organisms.

Numerous kits are available for detection of *E. coli* and coliforms. These include the Colifast system (Lysaker, Norway), which can be purchased in a single sample or multisample configuration, the CHROMagar ECC (CHROMagar Microbiology, Paris, France), Coliscan (Microbiology Laboratories, LLC, Goshen, Ind.), and the Colilert and Colisure systems (IDEXX, Westbrook, Maine). Among these systems Colifast advertises the fastest detection times (8 to 10 hours). It may be possible to speed the enzyme-assisted detection process via improved signal detection and optimized enzyme or enzymes (Nelis and Van Pouke, 2000). Among commercially available enzyme techniques for coliform or *E. coli* detection, Colisure and Colilert have been approved by the USEPA for testing drinking water (McFeters et al., 1997).

Another enzyme-related method involves direct reaction of an enzyme with the ATP present in a sample, resulting in luminescence (Lee and Deininger, 2001). Bioluminescence is viable for rapidly (on the order of minutes) detecting total bacterial count but does not provide information about specific bacteria. Only one on-line commercial monitor was identified in the course of this study: the Aquatrace System (Biotrace, UK). The device is available but has not been deployed at a water utility. Disadvantages of the device are its lack of specificity and anticipated high operating costs associated with consumable reagents.

Rather than detecting a molecule from a pathogen that has been labeled or bound to another substance, an experimental process called TIGER (triangular identification for genetic evaluation of risks) uses electrospray ionization and mass spectroscopy for directly measuring RNA weight (Stix, 2002). The RNA weight is then compared against a database of RNA weights and matched with the appropriate organism. This technique is not yet ready for commercialization, mainly because of problems overcoming interference by background organic material. Units will cost in excess of $200,000 when available.

IMMUNOLOGICAL METHODS. In immunological methods, an antibody is attached to a substrate and its response is monitored when it is exposed to a sample that may contain a target organism. Commonly used immunologically based systems are the enzyme-linked immunosorbent assays (ELISAs) and enzyme-linked fluorescent assays (ELFAs). The response of an antibody to a stimulus in the presence of appropriate enzymes may be generation of a spectrophotometric signal, generation of a fluorescent signal, or generation of a luminescent signal.

This signal may be detected manually or by an automated system. Detecting labeled organisms may be a time-consuming process (Ventner, 2000), and various means (e.g., flow cytometry or other means for automatically detecting a signal) are being developed to speed that portion of the detection process. Several vendors sell ELISA "readers" that automate the signal detection process. These readers usually are specialized spectrophotometers that assist people with relatively little training in immunological methods in interpreting results. Grayman et al. (2001) list the following water-related pathogens as having been detected by immunological methods: *Campylobacter jejuni*, *Klebsiella*, *Klebsiella pneumonia*, *Bacillus anthracis*, *Lactobacillus*, *Legionella pneumophilia*, *Salmonella*, *Listeria monocytogenes*, *Escherichia coli* O157:H7, *Staphylococcus aures*, *Pseudomonas aeruginosa*, *Shigella flexneri*, *Shigella* spp., *Streptococcus pneumoniae*, *Vibrio cholera*, *Clostridium difficile*, *Toxoplasmodium*, and *Chlamydia*.

ELISA or other immunoassays may be purchased as kits or as automated detection systems. Complete kits cost on the order of hundreds of dollars and are available from many sources. A complete laboratory setup for ELISA analysis would likely include facilities for storage of antibodies (typically antibodies must be preserved frozen), an area for preparing assays, and a device for reading assay results (usually a spectrophotometer). Numerous vendors contacted indicated that, even for the most exotic biological agents, antibodies are available and there are no controls on their distribution. A spectrophotometer for reading and analyzing ELISA results costs on the order of $6,500. ELISA assays can be performed simultaneously for a large number of samples (e.g., 96-well units are commonplace) and appear well suited to routine distribution system monitoring for biological agents such as *Cryptosporidium parvum* or *Giardia*. Tests of single samples with an ELISA kit are not economical. Kits range in cost from $200 (for most agents) to $500 (e.g., for *Bacillus anthracis* spore kits).

Research International (Seattle, Wash.) has developed and is marketing an immunologically based detection system that boasts parts-per-billion sensitivities to biochemical species, such as proteins, viruses, bacteria, and spores, and 15- to 20-min turnaround time (Hoyle, 2001). The device, called the Analyte 2000, was originally developed for the military and is capable of detecting toxins and chemical agents as well as biological contaminants.

EXPERIMENTAL AND NEAR-FUTURE DEVICES FOR DETECTION OF BIOLOGICAL AGENTS. Numerous techniques were identified as promising for contaminant detection but not yet ready. These techniques may not have been commercialized or may not have been applied to drinking water systems. Some of the more interesting and promising techniques are described below.

Pescovitz (2000) reported progress toward a “bioagent chip” that would be able to detect target organisms within seconds and at very low concentration. Unlike other immunological methods, the biochip uses antibodies resident on B-cells (as they are in the human body) aligned in microchannels and enhanced with a gene that makes them luminescent when responding to a substance. Pescovitz does not comment on the state of development of this device or speculate on when commercial versions might be available. In a subsequent article, Casagrande (2002) indicates that the sensor, still under development at MIT’s Lincoln Laboratory, is called *Canary*. A Lincoln Laboratory web site (http://www.ll.mit.edu/careers/seminars.html) indicates that Canary may prove capable of environmental monitoring and detecting biological warfare agents and that response times will be on the order of one minute.

An immunological sensor technology still in its infancy is detection of pathogens via amplification of the sound they make when they are separated from antibodies to which they are bound (Behar, 2002). Cambridge University researchers report succeeding in amplifying and detecting the sound of a virus (Herpes simplex) detaching from antibodies as a quartz crystal was vibrated at high frequency. It is projected that this technique will allow on-the-spot detection of viruses and bacteria (protozoans and other microbial pathogens are not mentioned). A company has been formed to develop and commercialize this invention.

FLOW CYTOMETRY. Flow cytometry provides an alternative to signal generation and processing methods that can detect only one or a small number of target organisms at a time. In flow cytometry, cells are labeled and passed single-file through a device that irradiates them with one or more lasers, causing scattering of the beams and possibly fluorescence of the organism. Each organism causes characteristic responses to the laser and is identified based on comparison of a given organism's response with known responses. A flow cytometry system ideally includes a front-end device for sorting cells for use in the cytometer and the cytometer itself. Cytometry has been shown adept at detecting *Giardia* and *Cryptosporidium parvum* (Hoffman et al., 1997) and is routinely used for detecting those organisms by European water utilities. Cell sorters are quite expensive (more than $100,000 in 1997), but Hoffman et al. report the per-sample cost of flow cytometry (including depreciation and overhead) to be less than that of immunofluorescent assays and their detection of *Cryptosporidium* and *Giardia* to be superior to immunofluorescent methods.

DETECTING SURROGATE INDICATORS OF LIVING BIOLOGICAL AGENTS. Particle counting may provide a means for on-line detection of pathogenic organisms introduced to a distribution system. Bridgeman et al. (2002) report success in deploying particle counters for monitoring filter performance but are dubious about the current state of the art in particle counters for monitoring *Giardia* and *Cryptosporidium*.

Particle counters rely on one of three sensor technologies (Van Gelder et al., 1999):

1. Light scattering
2. Light obscuration
3. Electrical sensing zone (electrical resistance–related) technology

The most natural use for particle counters in water treatment and distribution is monitoring filter performance. Even in that setting the use of particle counting is problematic, since dependable, consistent results may not be available from a single particle detector, much less different units. Deploying particle detectors in distribution systems would be difficult given the myriad reasons for potential changes in particle count at any location.

Another surrogate frequently mentioned for monitoring for living biological agents is chlorine residual. Chlorine residual may disappear as a result of gross contamination (with attendant demand) or intentional dechlorination in concert with introduction of a pathogenic organism, though the latter scenario appears far-fetched given its complexity.

Detecting chemical agents

Many chemical detectors currently used in laboratory analyses may not be appropriate for rapid detection or continuous monitoring because they are large, require extensive sample preparation and manipulation, or require highly skilled operators. Fortunately the development of both faster and more sensitive laboratory equipment of virtually every type has taken place over the last decade and is yielding a steady stream of portable, accurate, automated monitors. The detector types receiving the most attention in recent technical literature are biomonitors, biosensors, ion-selective probes, electronic noses and tongues, and miniaturized versions of equipment traditionally constrained to laboratory work. Early warning system studies provided a starting point for identifying viable chemical detectors for distribution system monitoring or analysis. Brosnan (1999) identifies ion chromatography, liquid chromatography, gas chromatography, and biomonitors (systems based on the response of living organisms) as "medium end (costs in the tens of thousands of dollars)" approaches to monitoring and ion probes, UV detectors, smell bells, and immunoassays as "low end" viable approaches to early warning monitoring.

TOXIMETERS. The "canary in the coal mine" has emerged as a popular detection technique for monitoring general water toxicity. Toximeters have been developed based on many organisms (listed in Table 3), each with its own requirement for upkeep (food, dechlorination, etc.) and its own signal (aversion behavior, luminescence, electrical signal, etc.) (Brosnan, 1999; Grayman et al., 2001). Grayman et al. (2001) points out several limitations of biomonitors, including the ability of a single species to detect all important contaminants, complex signal processing requirements, response time of organisms to contaminants, potential for false positives (since organisms respond to many stimuli), and operational and capital costs.

Table 3 Biomonitoring schemes

Species	Response	Response Measurement
Fish	Avoidance behavior	In avoiding a stimulus fish touch a sensor located in a strategic tank location.
Mollusk	Shell opening/closing	Electromagnetic sensors produce a signal whose magnitude depends on the distance between the emitter and receiver.
Daphnia	Survival (static tests) or change of location (continuous testing)	The number of interruptions of a light beam are counted. If the interruption rate declines rapidly an alarm is signaled.
Algae	Metabolism change or death	Decreased photosynthesis leading to increased fluorescence
Bacteria	Metabolism change or death	Reduced fluorescence or reduced respiration with attendant change in dissolved oxygen

BIOSENSORS. A schematic diagram of biosensors adapted from Rogers and Gerlach (1999) is shown in Figure 2. At its simplest, the biosensor consists of (1) a biological element that captures or reacts with a target analyte and (2) a means for creating a signal based on the activity of the biological element. Many systems are commercially available for applications other than water quality monitoring, but only one biosensor whose use is intended for water quality monitoring, Research International's Raptor, was identified in the course of this project. Rogers and Gerlach indicate that biosensors have been developed and employed in detection of organophosphate, organochlorine and carbamate pesticides, BTEX, explosives, chlorophenolscyanide, sulfite, and heavy metals, among other compounds.

Research continues to produce immunoassay formats amenable to use in biosensors (Sapsford et al., 2002), and the number and sophistication of devices available for drinking water monitoring in the near future should increase. Development of biosensors has led one observer (Turner, 2000) to predict a "microanalytical revolution" that will complement the last decade's microprocessor revolution.

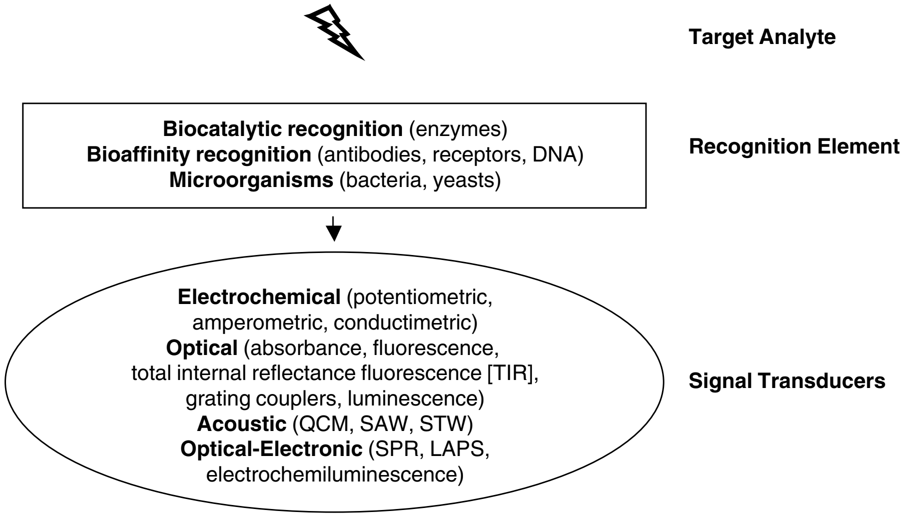

Figure 2 Biosensors schematic (Rogers and Gerlach, 1999)

ION-SELECTIVE ELECTRODES. In the last four years ion-selective electrodes (ISEs) have emerged as an extremely accurate choice for monitoring ionic species in water (Bakker and Pretsch, 2002). Though ISEs have been in common use for decades, their improved sensitivity, miniaturization, and incorporation in chemical sensor arrays have improved their utility. Specifically, the ISEs can measure heavy metal concentrations (lead, cadmium, and copper are mentioned in the article) as well as if not better than current laboratory techniques and offer the advantage of measuring only free ionic species. The greatest limitation of ISEs is the interference of nontarget ions with the probes. Grayman et al. (2001) add bromide, chloride, cyanide, and nitrate to the list of species that might be monitored with ISEs. Many companies produce ISEs; they are relatively inexpensive, costing about $300 to $500.

INFRARED ABSORPTION INSTRUMENTS. As with other detector types, infrared sensors are in common use beyond drinking water monitoring and researchers are exploring their utility for drinking water monitoring. Infrared sensors measure the amount of infrared radiation absorbed in a sample or at the surface of a probe. Sensors differ in the wavelengths of radiation utilized and how resulting absorption spectra are analyzed and interpreted. Mizaikoff (2003) notes the need for improved monitoring capabilities in natural and engineered water systems and forecasts that infrared sensors will soon be able to provide a means for on-line and in situ monitoring of numerous compounds and classes of compounds.

A number of studies have employed infrared sensors in rapid detection of compounds of significance in drinking water. Roy and Mielczarski (2002) configured a commercially available infrared optical sensor for ppb level detection of chlorinated hydrocarbons in water. Experiments were performed on pure water samples spiked with six contaminants. The authors declared the sensor easy to use, rapid (results within five minutes), and capable of producing results without preconcentration. A limitation the authors noted was the diffusion of the target analyte to the sensor surface, leading to the recommendation that optical sensors not be employed directly in environments where laminar or low-mixing conditions exist. No mention was made of how the sensor might perform in analysis of a more complex sample.

There are a number of potential drawbacks of infrared optical sensors. The ability of the sensors to provide reliable information within the confines of the complicated background of the water matrix is unproven. As with other optical devices, fouling would degrade the performance of the sensor. Transport of contaminants to the sensor may pose a limitation for some sensors and in some flow conditions. And finally, infrared sensors are considerably less sensitive than GC or other classical laboratory techniques.

FIELD-PORTABLE LABORATORY INSTRUMENTS. A number of detector companies have developed "laboratory in the field" versions of detection equipment, mainly in response to military detection needs. These devices are now available to the general public. Syagen, a California detectors company, has developed a portable (30 lb) GC/TOFMS (Time of Flight Mass Spectrometry) unit capable of both liquid and air sampling (Syage et al., 2001). The current version detects single compounds and in the near future the authors expect the device to have the capacity to rapidly scan a database in detection of unknown compounds. Other lab-in-the-field-type detectors have been developed by Electronic Sensor Technology, Perkin-Elmer's, Sentex System, Bruker-Franzen Analytical Systems, Bruker-Dalton, and Infinicon Inc.

CROSS-REACTIVE CHEMICAL SENSOR ARRAYS (ELECTRONIC NOSES AND TONGUES). Just as the brain processes nonspecific signals from sensors in the nose or tongue into useful composite information, cross-reactive chemical sensor arrays are composed of multiple sensors, each of which may respond to multiple chemicals or classes of chemicals. The sensor evaluates water quality based on the composite output of the sensors. Potential advantages of cross-reactive sensors over traditional “lock-and-key” (single target analyte or property) sensors are (Albert et al., 2000):

- A single sensor array can be configured to detect multiple analytes of interest.
- Sensor elements of an array are generally simple, and can operate at the pressures and temperatures typically encountered in water treatment plants and distribution systems.
- The signal-processing component of the sensor array can be “trained” to identify additional compounds or to differentiate between target analytes and similar compounds.

Cross-reactive chemical sensor arrays are available for gas phase and liquid phase monitoring (electronic noses and electronic tongues). Like other sensors described in this chapter, the signal and signal processing differ between electronic nose models. Grayman et al. (2001) list the primary signal generation and processing mechanisms as sensor acoustic wave (SAW, more commonly called surface acoustic wave), quartz microbalance (QMB), metal oxide (MOX), and polymer composite sensor (PCS). Cross-reactive chemical sensor arrays became practical and commercially available only recently (mid-1990s) as a result of advances in sensors and other components employed in the devices (Fenner and Stuetz, 1999). As a new technology, their viability as practical drinking water monitors is yet to be determined, though they have been assessed as positive additions in wastewater treatment (Dewettinck et al., 2001; Bourgeois and Stuetz, 2002), and are in common use in food quality control and other applications. There are obvious dual-use implications if they are chosen for use in an on-line contamination monitoring system.

There are published accounts of electronic noses being used in both water and wastewater treatment plants, though their use in wastewater plants is more common. For example, an experimental study illustrated the ability of an electronic nose (the ProSAT sensor array module, Marconi Applied Technologies) to detect potentially harmful organic compounds in raw wastewater (Bourgeois and Stuetz, 2002). Dewettinck et al. (2001) explored the viability of electronic noses for monitoring volatile suspended solids in wastewater treatment for the purpose of process control. The authors demonstrated the concept, though they point out that further refinement of the process and calibration of the sensor array are needed. The only published report of the application of electronic nose technology in drinking water treatment entailed detection of volatile hydrocarbon contamination in raw water (Fenner and Stuetz, 1999).

Electronic tongues (Jensen, 1999) use multiple simultaneously operating chemical sensors to detect flavor elements such as a human tongue might taste (acidity, saltiness, etc.). As of the writing of this chapter, commercial electronic tongues are available, though none has been tested in a drinking water plant or distribution system. A manufacturer representative indicates that an electronic tongue for use in drinking water monitoring would have to be custom-designed based on the target analytes and the anticipated background solute concentrations. A significant training period (perhaps a year) would be needed to develop an understanding of the sensor's output and to calibrate the device.

SURROGATE MEASURES. A number of devices capable of simultaneous on-line measurement of multiple water quality parameters can be purchased. These devices may not be able to detect conditions related to the presence of any of the priority contaminants listed in phase 1 of this project, but may provide useful information about fluctuations in water quality that could help operators infer that contamination has occurred. For example, Dascore Inc. (Tallahassee, Fla.) offers its Sixth Sense and CT-Sense probes capable of simultaneous remote monitoring of chlorine (without addition of reagents), DO, pH, temperature, conductivity, and redox potential. Hach has recently commercialized a panel for distribution system monitoring that can include a TOC analyzer (based on UV absorbance) in addition to conductivity, pH, temperature, chlorine, and other parameters. A number of utilities have reported pilot programs under way for monitoring with multiparameter sensors (e.g., Schreppel, Tangorra et al., 2002). Particle counters (described above) may also be employed in distribution system monitoring. However, differentiating between "normal" changes in particle count or size distribution and worrisome changes may make their use in distribution system monitoring impractical.

RECOMMENDATIONS

PCR for Biological Agent Monitoring

At present, the only available techniques that detect the diverse set of biological agents described above are PCR and immunoassays. Comparing PCR and immunological assaying, PCR offers far greater sensitivity and flexibility, though with slightly higher instrumentation cost and much higher educational requirements for laboratory personnel. Given the low sensitivity of ELISA assays (usually 10^6 organisms or more are required to generate a strong signal), PCR emerges as the best lab upgrade a water utility laboratory can implement for improving its ability to screen water for biological contaminants.

How PCR would be used

Though developments are under way to enable on-line or frequent PCR monitoring, available PCR equipment is constrained to laboratory analysis of discrete samples. So, most probably, PCR would be used for analysis of samples drawn from the field after known or suspected contamination or intrusion. For example, utilities might want the capability to screen for priority agents if a credible threat were to emerge and the utility required assessment faster than coordination with law enforcement or other laboratories would provide.

To establish a credible PCR capability, a utility's laboratories would require instrumentation (described below), at least two staff members, and a stock of reagents for agents identified as potential threats. Stand-alone PCR systems (e.g., the DuPont BAX® system) are available and might make PCR analysis easier. But in light of the diversity of agents a utility might need to detect and the possibility that reagents would not be available for use in stand-alone systems, we recommend that utilities purchase PCR components individually. Doing so will allow greater flexibility in analysis and an easier upgrade of system components—at significantly lower reagent costs. A trained supervisor (probably with master's-level education) would be able to develop methods for detection of all priority biological agents and determine which reagents should be purchased. There is no restriction on obtaining primers, even for CDC category A agents, and primers and other material can be kept as long as five years if stored properly (frozen). A laboratory technician with bachelor's-level education in biology could perform actual PCR amplification and analysis after limited training.

PCR will provide more than contaminant detection

As a fast and sensitive technique for detecting organisms, PCR could be used for more than just security screening. Routine analyses and monitoring (e.g., for *Cryptosporidium parvum* in source water) would be improved or facilitated if PCR were used. Developments in PCR have led to its ability to perform quantitative analysis. For example, "real-time" PCR is now available, offering a means for estimating the original quantity of DNA in a sample. The more water utility laboratories use PCR, the more useful it will likely become. PCR's potential will be fulfilled as laboratories establish PCR's sensitivity in source and finished water matrices and develop optimized methods for detection. In this regard, PCR might be regarded as analogous to GC/MS in the 1970s, at which point it started to emerge as an analytical tool in water quality assessment. Moreover, as a sensitive method, PCR offers the opportunity of better characterization of background concentrations of waterborne living organisms. Routine use of PCR has the potential of providing a better understanding of biological water quality, which could lead to improvements in disinfection, reduction of regrowth in distribution systems, and characterization of source water.

PCR has limitations

The most significant limitations of PCR for use at a utility's laboratories are sample preparation requirements and PCR's inability to distinguish between living and dead organisms. In conventional PCR, sample sizes are generally between 25 and 100 μL and inhibitory substances must be removed. Thus, sample preparation requirements (filtration, resuspension, separation, and possibly other steps) may be time-consuming and may limit the accuracy of the tests. Also, PCR does not distinguish between materials from living and dead cells, so PCR is often performed after culturing, resulting in slower and more complex analyses. As PCR gains wider acceptance in the drinking water utility community, it is expected that techniques will be developed to mitigate these difficulties.

ELISA assays—a lower-cost alternative

If a utility does not believe benefits merit the cost of adding PCR capability to its laboratory, ELISA assays offer a cheaper though less sensitive capacity for priority biological agent detection. As mentioned above, ELISA assays do not produce strong signals unless many organisms (on the order of 10^6) are present. Thus, for the assays to be useful, samples would need to be drawn close enough to the contaminant's release, soon enough after the release, and in sufficient quantity for enough organisms to be present. If those requirements are not met, culturing must be done before the assay is performed. Setting up a laboratory for ELISA assays for priority agents would be relatively inexpensive (around $6,500 for a device to read assays and between $200 and $500 for each antibody). Distributors indicate that antibodies are available for virtually any agent and can be purchased and used without any restriction. Antibodies can be stored (frozen) for as long as two years. Equipment and techniques are simple and could be used by a technician with bachelor's-level education and training of only a few days.

Chemiluminescent or Bioluminescent Field Test Kit and On-line Monitoring for Chemical Agents

We recommend that utilities augment their capability for field detection of toxic agents and begin a program of distribution system monitoring. For investigating suspected contamination events, the cheapest and most comprehensive means is field test kits using either chemiluminescent reagents or bioluminescent assays. For on-line monitoring a multi-parameter sensor array is inexpensive, offers utility beyond contamination detection, and will yield valuable information about water quality (though not specifically about the contaminants identified above).

Chemiluminescence for investigating suspected contamination

The only on-line monitor that detects all the chemical contaminants listed above is the toximeter. However, these devices are large, have significant operational requirements, and are quite expensive. So rather than on-line monitoring, we recommend field-testing after reports of intrusion or suspected contaminant introduction. Numerous kits are available for this purpose, with the least expensive based on chemiluminescence (emission of light due to reaction of target analyte with appropriate reagents) or bioluminescence (emission of light by select bacteria). These kits can be used at the site where a sample is taken and will produce results within minutes. Applications would be rapid assessment of water after a suspected release or tracking of contaminants in the distribution system. Positive results would lead to further analysis by the utility or turning the investigation and analysis over to law enforcement officials. Operation of the equipment is simple and could be performed by laboratory technicians after minimal training. Periodic (e.g., annual) demonstration of the equipment would likely be sufficient for maintaining a trained laboratory staff.

Several relatively inexpensive kits are available (e.g., the Checklight system [Checklight Technologies, Kiryat Tivon, Israel], the Eclox system [Severn Trent Services, Fort Washington, Pa.], and the Microtox system [Strategic Diagnostics, Newark, Del.]). The Eclox system (sold in the United States by Apprise Technologies) is based on chemiluminescence and detects carbamate, organophosphate and organochlorine pesticides, cyanide, mercury, lead, arsenic, and chlorine (as well as other substances) at the ppm level. The Microtox and Checklight systems are based on bioluminescence (freeze-dried bacteria are introduced into solution with the sample, following which the luminosity of the solution is measured). Costs for complete chemiluminescence and bioluminescence kits range from around \$3,000 to \$18,000 and recharge kits can cost between \$50 and \$700, depending upon the quantity purchased.

Pave the way for on-line monitoring with a sensor array and UV detector

Based on discussions at the PWD/Drexel University detectors workshop (May 2003), the USEPA/Rutgers CIMIC detectors workshop (November 2002, Newark, N.J.), and personal communications, water utilities are reticent to employ on-line sensors for a number of reasons. These include poorly characterized background, leading to difficult data analysis, a dearth of data analysis tools, and operational concerns such as security, fouling, and access. To overcome these difficulties we recommend that utilities undertake a pilot distribution system monitoring program using relatively inexpensive sensors, with an eye toward understanding and improving distribution system water quality as well as contaminant detection. Numerous means are available for on-line monitoring, including multiparameter sensors, UV spectroscopy, and cross-reactive chemical sensor arrays. Cross-reactive chemical sensor arrays (electronic tongues) are still experimental and would likely produce data more complicated than merited by a pilot program. So we recommend a UV detector be deployed in tandem with a multiparameter sensor array. Operational experience will expose the sensors' tendency to foul, their reliability, and their dynamic ranges.

Develop a network of laboratory partners

Though we cannot justify the purchase of expensive, specialized detection equipment (such as HPLC or variants of GC/MS) by individual utilities, we do recommend that utilities develop relationships with regional laboratories owning those instruments. A large-scale terrorist attack or other emergency would certainly stretch any utility's and other agencies' analysis and response capacities. If health department, law enforcement, USEPA, water utility, and other officials are competing for a limited number of detectors, the water utility may not be assigned top priority. Developing understandings and relationships with industrial laboratories, university laboratories, public health laboratories, and commercial laboratories will make additional equipment available and prevent bottlenecks. Such an arrangement could be done individually, in concert with other utilities in the region, or with the coordination of organizations such as the American Water Works Association.

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CHAPTER · 30

On-line Real-Time Monitoring: Peace of Mind?

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SUMMARY

During the past five years, distribution systems have been responsible for 45% of outbreaks of waterborne disease (*AWWA Journal*, Sept. 2001). The looming threat of bioterrorism and the vulnerability of the distribution system to attack have pressed the water industry to investigate the possibility of using on-line, real-time monitoring along with rapid screening of the water for toxic substances. These systems may prove to be a valuable monitoring tool to the industry, to water quality, and to the safety of the American public.

Mohawk Valley Water (MVW) heightened its concern for the vulnerability of its distribution system after the September 11 terrorism attacks. The MVW system has been classified as high risk due to the fact that it serves a population greater than 125,000 people and has two open finished water storage reservoirs located in the distribution system. Mohawk Valley Water's 10-year capital plan includes additional covered water storage to eliminate these open reservoirs, however, a more immediate plan to safeguard the system was recognized as an important priority.

Along with basic security measures such as site security, increased patrols of the distribution system, surveillance cameras, and ID entry systems, MVW has chosen an on-line monitoring system utilizing the Dascore "Six-Cense" ceramic chip that uses Siemens CENSAR (Chemical ENvironmental Sensing ARray) technology. A network of remote monitoring stations has also been installed at strategic points of the distribution system. Along with these systems several rapid screening processes designed to monitor for toxic substances have been evaluated. These include the Checklight Ltd. Toxicity Testing System, a rapid method of

quantification of viable bacteria using an ATP assay (New Horizons Diagnostics), immunomagnetic separation and flow cytometry for rapid detection of pathogenic microorganisms (Advanced Analytical Technologies Inc.), and biological (fish) monitors. This chapter describes and evaluates the use of each of these techniques in drinking water.

INTRODUCTION

The design and operation of an early warning system should be viewed as an integral part of the operation of a water system (Deininger et al., 2002). This AwwaRF research project centered on the design of early warning and predictive source-water monitoring; however, the guidance for the design of such source water systems can also apply to predictive distribution monitoring systems. These systems should include the following elements:

- A way to detect the presence of a contaminant
- A way to confirm the presence of a contaminant
- Procedures to manage the efforts associated with the contamination event
- A communication network for information related to the contamination event
- Ways to respond to the contamination event and minimize its impact on the public

The most important element of an on-line system is its ability to detect the possible occurrence of a contaminant. The system should be designed and operated so that there is sufficient lead time to react to the event and actions can be taken to minimize the impact of the contaminant on the distribution system water.

Lately there have been several advances in monitoring technology. Some of this emerging technology has become commercially available. However, many of these advances are not being made by the water industry but by the food and beverage industry and the military to counter threats of biological and chemical warfare. The water industry needs to place greater emphasis on on-line and rapid detection methods and further research should be encouraged specific to the water industry.

MATERIALS AND METHODS

Remote On-line Monitoring

In December 2001, MVW began planning an on-line distribution system monitoring system. The Dascore "Six-CENSE" Censar utilizing Siemens CENSAR (Chemical ENvironmental Sensing ARray) technology was chosen to be installed at representative points in the distribution system, providing an electronic network monitoring surrogate water quality parameters. CENSAR technology was developed for long-term monitoring of coastal waters. Research led to the use of ceramics as a substrate for the chip since the ceramic material was resistant to corrosion, biofouling, and abrasion. The chip is layered with gold onto the ceramic base and is patented in both the United States and England. All materials in the chip are inert and approved by the FDA and NSF for contact with potable water.

Further research led to the development of the CENSAR probe, which was designed to measure water within a pressurized distribution system. Yorkshire Water, UK, teamed with Siemens Inc. to validate the concept. Yorkshire Water produces over 475 mgd of potable water, operates 116 water treatment plants, and serves over 4.6 million customers. There are over 150 of the Six-CENSE units installed with over 18 months of valid data collected, illustrating the validity of the concept, the robust design, and the user-friendly operation of the technology. The CENSAR chip provides the ability to detect potentially harmful changes in water quality throughout the water distribution and storage systems, minimizing the impact on the water users and saving time and money in implementing effective solutions.

The Six-CENSE monitors six critical water quality parameters. A 1-inch-square ceramic chip forms the foundation for the technology used to measure the six parameters, which include chlorine or chloramines, dissolved oxygen, pH, conductivity, redox/ORP, and temperature. The chip is based on standard electrochemical methodologies that eliminate the requirements for reagents in the measurement of chlorine, glass for the measurement of pH, and membranes for the measurement of dissolved oxygen.

The Six-CENSE technical specifications are as follows:

Chlorine: direct and reagent-free measurement

The sensor utilizes the electrochemical reduction of chlorine species at a micro-electrode. A proton generator electrochemically reduces the sample pH to below 5, allowing the chlorine-sensing electrode to measure the CL_2 and HOCl.

- Range: 0–5 mg/L
- Sensitivity: <0.01 mg/L
- Accuracy: +0.04 mg/L or 5% of reading, whichever is greater

Dissolved oxygen: membrane-free measurement of DO

The dissolved oxygen is measured using an electrochemical voltametric technique. The current flowing into the working electrode is proportional to the concentration of dissolved oxygen in the water. Electrolysis is used to acidify the surface of the sensor as an antifouling measure, and the geometry of the sensor reduces sensitivity to flow. These two features eliminate the need for a membrane, making the sensor more robust than conventional DO sensors.

- Range: 0–20 mg/L or 0–200% saturation
- Sensitivity: <0.1 mg/L
- Accuracy: +0.1 mg/L or 5% of reading, whichever is greater

Temperature

A solid-state temperature sensor allows for the direct reading of temperature.

- Range: 0–+50°C
- Sensitivity: <0.1%
- Repeatability: +0.1%
- Accuracy: +0.25°C or +0.1% of reading, whichever is greater

Conductivity

- Range: 0.1–10.0 µS/cm
- Sensitivity: < 10 µS/cm
- Accuracy: +10 µS/cm or +1% of reading, whichever is greater

pH

The electrode consists of a metal oxide deposited on a printed base material. A pH measurement is obtained using a potentiometric measurement technique with respect to the reference electrode.

- Range: 2–12
- Sensitivity: <0.1 pH
- Repeatability: +0.1 pH
- Accuracy: +0.5 pH

The pH reference electrode is a silver/silver chloride type with a drift < 5 mV in six months. The operational life is rated for a minimum of six months' continuous operation.

Redox/ORP

The redox electrode, constructed from platinum ink, will respond to all ions present in the solution. Each ion contributes (positive and negative potentials) to the overall potential generated by the water. The measurement of the oxidation/reduction potential utilizes a potentiometric measurement.

- Range: –1.4 to 1.4V
- Sensitivity: <1% of range
- Repeatability: +1% of range
- Accuracy: +1% of range

The Six-CENSE utilizes a 37 mm stainless steel probe designed for direct insertion into water mains from 2 to 36 in. in diameter through a 1.5- or 2-in. valve or corporation stop. Continuous operating pressures of up to 230 psi are allowed.

The data output of the probe is available in various output formats to integrate into SCADA systems, and telemetry options can be used for continuous monitoring at remote sites. Setup of the probe is easily accomplished with single point calibrations and maintenance costs are relatively low.

Remote Sampling Station Network

Mohawk Valley Water has over 600 miles of distribution mains that travel from areas of high population to sparsely populated remote areas.

Gaining access to reliable taps for monitoring purposes is often a challenge. A network of Kupferle Foundry Eclipse Sampling Stations is being installed at strategic points in the distribution system. This network will enable the Water Quality Department to get a better picture of the quality of the water in the remote areas of the distribution system.

Toxicity Testing and Bioluminescence-based Assays

The bioassay developed by Checklight utilizes a highly sensitive variant of the luminescent bacterium *Photobacterium leiognathi* that allows the detection in water at levels below milligrams per liter for a diverse group of toxicants (Ulitzur et al., 2002).

The use of luminous bacteria for toxicity assessment has advantages that have been scientifically validated (Bulich and Isenberg, 1981; Kaiser 1998). The bacteria are self-maintained luminescent units that emit high and steady levels of luminescence (490 nm). The light emitted by several hundred bacterial cells can be detected with a luminometer. Biological toxicants and chemical agents that affect cellular respiration, protein, or lipid synthesis alter the level of luminescence. Likewise, agents that alter the integrity of the cell membrane have an effect on the amount of luminescence produced by the bacteria. Many kinds of toxicants, such as pesticides, herbicides, chlorinated hydrocarbons, and heavy metals, exert a measurable effect on the luminescence of the bacteria. By comparing the levels of luminescence produced by bacteria added to a water sample suspected of being toxic with those of a clean control water sample plus the bacterial culture, low concentrations of a broad range of toxicants can be detected.

Plant effluents, leachates, and soils have been tested using bioluminescent-based assays in the past (Bitton and Koopman, 1992). However, because of limited sensitivity to some toxicants these systems were not widely used as early warning systems for drinking water toxicity. The bioassay developed by Checklight was evaluated and found to detect levels smaller than milligrams per liter of cationic heavy metals, pesticides, polycyclic aromatic hydrocarbons (PAHs), and chlorinated hydrocarbons within 20–45 min (Ulitzur et al., 2002).

The luminescent bacteria are a variant strain of *Photobacterium leiognathi* Eilat-1, isolated from the Red Sea (Katznelson and Ulitzur, 1977). The toxicity test kit contains these bacteria in a freeze-dried state, and once hydrated the bacteria promptly become luminescent. An aliquot of the bacterial culture is added to a tube containing the water in question in combination with a pro-metal buffer (favors the detection of heavy metals). A second aliquot of bacteria is added to a tube containing the water sample and a pro-organic buffer (favors the detection of organic pollutants). The buffers were developed by Checklight Ltd. to enhance the sensitivity of the test to a wide range of toxic agents with different modes of action. The pro-metal and pro-organic buffers are strong buffers that minimize the artifactual toxic effects of extreme pH encountered when using other assays that have only NaCl solutions as the diluents.

Another advantage of the Checklight system is the stability of the bacteria under ambient temperature conditions. Studies have revealed that the bacterial cultures of *Photobacterium leiognathi* are not as temperature sensitive as other toxicity screen tests that utilize the bacteria *Vibrio fischeri*. Because of the stability of the kit's bacteria the Checklight toxicity kit can be shipped at ambient temperatures, eliminating the need for refrigeration or freezing during shipment, significantly reducing testing costs. The test is also sensitive over a wide temperature range of 18°–25°C.

Rapid Quantification of Viable Bacteria Using an ATP Assay

The heterotrophic plate count (HPC) is used to estimate the number of heterotrophic bacteria that form colonies on agar plates. The present Standard Method currently used takes 48 hr to seven days to obtain a result. Many techniques available for rapid bacterial analysis are expensive, require expensive equipment and expertise, and are not able to determine viability.

The determination of adenosine triphosphate (ATP) using a bioluminescence assay (New Horizons Diagnostic's Profile-1 System) may solve the problems of determining the viability of bacteria (Trudil et al., 2000; Lee and Deininger, 2001). The assay quantitates ATP, an energy-rich molecule found within all living cells. The assay is based on a reaction between the enzyme luciferase, the substrate luciferin, and ATP. Light is emitted during this reaction and can be measured quantitatively and correlated with the ATP quantity extracted from the bacteria. The Profile-1 System was developed to rapidly identify generic bacteria levels in food, water, environmental, and work surfaces and can be used to detect spores. It has been used by the USDA, the US military, and the Canadian Food Inspection Agency and has been field-tested for drinking water by the University of Michigan. USDA-published data indicate that results correlate to aerobic plate counts with r=~0.92.

Drinking water is sampled using a sterile syringe. A Filtravette™ (New Horizons Diagnostics, NHD), which is a combination of a 0.45-micron pore size filter and a cuvette, is placed into a 13-mm Swinex filter holder (Millipore Corp.). The filter holder is screwed into the syringe and the water sample is pushed through the filter. The Filtravette is then removed and placed on sterile blotting paper. A somatic cell releasing agent (NHD) is added to lyse all nonbacterial cells and to release ATP. A specially converted 3-mL syringe is used to apply air pressure to remove nonbacterial ATP through the filter. The Filtravette retains bacteria on top of the membrane and the bacterial ATP remains within the bacterial cell membranes. The Filtravette is then placed into the microluminometer and the bacterial cell-lysing agent (NHD) is added to lyse the bacterial cells retained on the membrane. The released bacterial ATP is mixed with 50 µL of luciferin-luciferase (NHD). The light emission is recorded after 10 seconds and the unit is called an RLU (relative light unit). The result is expressed as RLU/mL by dividing the RLU by the filtered water volume. The RLUs are proportional to the amount of ATP, and the amount of ATP is proportional to the amount of viable bacteria.

Lee and Deininger have determined that the New Horizons microluminometer was able to detect ATP as low as 0.2 pg. It is known that the average ATP content in one bacterial cell is 10^{-15}g (1 fg) (Crombrugge and Waes, 1991). Deininger concluded that 0.2 pg corresponds to 200 bacterial cells, which is the sensitivity of the ATP in terms of bacterial cell numbers and is approximately equivalent to 4 cfu/mL. For drinking water, the level of concern for bacteria is 500 cfu/mL; therefore the level of bacteria can be accurately detected using the ATP bioluminescence method.

Biological Monitors

Organisms such as fish, clams, mussels, daphnia, algae, and bacteria have been used as biomonitors. Prior to September 11 there were very few biomonitors in the United States. However, since that time, water systems have been investigating their usefulness. While biological monitoring systems that rely on a single species have limitations, they may prove useful when used in conjunction with other monitoring methods.

Fish have been used as monitors of water quality for centuries. The *Book of Exodus* 7:20–24 describes an incident, “And all the water that was in the river turned to blood. And the fish that were in the river died.”

A simple fish monitor that utilizes the “avoidance principle” (Grayman et al., 2001) is being constructed in the laboratory at MVW’s treatment plant. The monitor is based on the fact that fish will swim away from water that is contaminated with toxic agents. Five tanks connected in series are stocked with native fish. The fish are fed in the tank that receives the treatment plant intake water first. Fish tend to stay in the tank where they are fed, but if they sense a contaminant they begin to move downstream to the other tanks. If the fish in tank 3, 4, or 5 exceed the number of fish in tanks 1 and 2, treatment plant operators will be alerted to a possible contamination event. Presently, manual feeding and visual observation of the fish by on-duty water treatment operators are used to assess the well-being of the fish. In the future, imaging and/or alarms may be added to improve the system.

Immunomagnetic Separation Methods and Flow Cytometry

Routine water quality monitoring techniques using flow cytometry are just over the horizon. Flow cytometry has traditionally been used in hospital laboratories for the identification of bacteria in clinical samples of body fluids. The food and beverage industry has again pioneered the development of this technology for other uses, as they must detect pathogens that enter their processes and find their way into the finished product. Flow cytometric methods are rapid and quantitative and can be versatile, since many methods can be combined, such as nucleic acid probes and immunofluorescence. They are also rapid and may be used to monitor viability (Grayman et al., 2001).

The RBD2100, an instrument manufactured by Advanced Analytical Technologies Inc. (Ames Iowa), is a low-level microbe detector utilizing the principles underlying flow cytometry. The use of the instrument in the water industry has been evaluated for the identification of *Escherichia coli* O157:H7 and *Cryptosporidium parvum*.

Smith and Rice (2000) described the rapid determination of *E. coli* O157:H7. The organisms were separated using immunomagnetic separation, labeled with fluorescent antibodies, and injected into the instrument. *E. coli* was identified and quantified by measuring the fluorescent signal that resulted from the bacteria and the bound labeled *E. coli* antibody passing through the laser in the flow cell. Only *E. coli* O157:H7, with the bound specific antibodies, will fluoresce in the flow cell. It is the specificity of the antibody/antigen reaction that makes the method specific for *E. coli* O157:H7 and allows for low detection limits.

The AATI RBD2100 flow cytometer has also been evaluated for detecting *Cryptosporidium parvum* in drinking water (Lasky and Chen, 2001). The cysts are captured using immunomagnetic separation techniques, labeled with CY5 anti-*Cryptosporidium* antibodies, and analyzed by the RBD2100 flow cytometric methods.

RESULTS

Remote On-line Monitoring

The Dascore Six-CENSE monitoring chip was installed at a representative point downstream of the Hinckley Reservoir Treatment Plant for a trial period. Readings from the probe were sent to the SCADA system of the treatment plant. Following installation of the probe, several problems were encountered. Due to the low conductivity of our upland Adirondack water, the chip developed some scaling problems faster than would be expected. After investigation, it was determined that the probe needed to be inserted farther into the water flow of the main. Following the adjustments, expected readings were obtained from the Six-CENSE. Several other communication problems occurred, but after investigation by both Siemens and Dascore new software was installed, greatly improving communications between the chip and the treatment plant SCADA system.

To evaluate the Six-CENSE probe a Hach pH meter and Cl-17 chlorine analyzer were installed at the same location as the Six-CENSE probe. Readings from these meters have followed very closely the readings obtained from the probe. A Hydro-Lab water quality probe was used in parallel with the chip to confirm water quality parameters successfully.

Figure 1 is a graph generated by the Six-CENSE chip in response to an alteration in chemical addition at our treatment plant. Lime and soda ash are added for corrosion control purposes following filtration and chlorination. Time zero (t_0) represents when the chemical feed was discontinued for these two chemicals. At time t_1, the Six-CENSE probe located downstream of the treatment plant sensed this change and registered a drop in both pH and conductivity.

Following this successful testing period, the Six-CENSE probes are now being installed at representative points within the distribution system.

A source water monitoring system by Dascore will also be used in a trial by MVW to monitor source water quality. The River-CENSE is an instrument built to monitor BOD, COD, TOC, ammonia, TSS, pH, color, turbidity, conductivity, temperature, and selective ions such as nitrate or ammonium. It is a stand-alone unit about the size of a 16-ft^3 refrigerator that uses IR and UV absorption for the measurements. It has a built-in sample pump and the lines are self-cleaned with a biocide with no side streams. The River-CENSE can store up to 30 days of data and has a 4–20 nA output or RS 232/RS 485 that is connectable to SCADA.

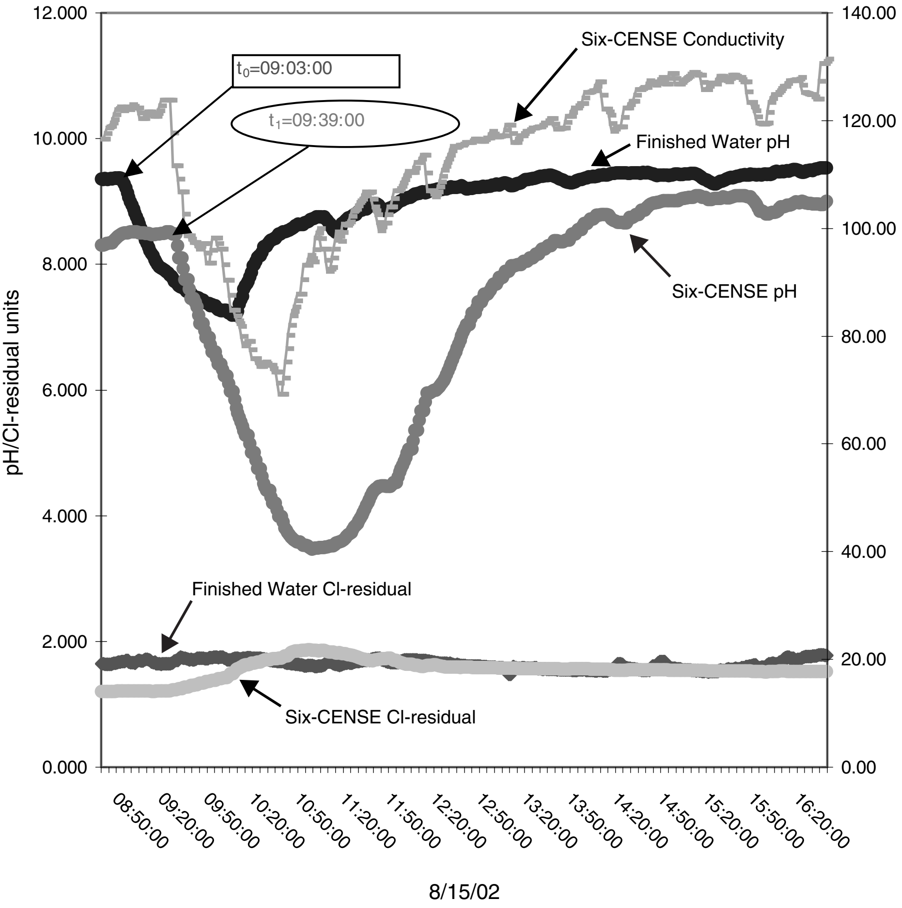

Figure 1 Six-CENSE chip graph in response to an alteration in chemical addition

Sampling Station Network

Ten sampling stations have been installed at remote points in the distribution system, allowing for water quality monitoring in areas that are not easily accessible. Capital plans include the installation of more sample stations each year, creating a web of water quality monitoring stations throughout the entire distribution system.

Toxicity Testing

Toxicity testing has been phased into the daily routine monitoring performed by the MVW Water Quality Laboratory. Controls are run with each testing sequence and matrix spikes using pesticides, herbicides, and metals are routinely performed. MVW has open finished water storage reservoirs within the distribution system, and toxicity testing is performed daily on the effluent from these reservoirs as well as representative points throughout the distribution system.

ATP Assay of Viable Bacteria

The ATP assay of viable bacteria has also been phased into the routine monitoring program of the MVW distribution system. The procedure has been compared to the Standard Method heterotrophic plate count (HPC) method (48-hr incubation at 35°C) and the heterotrophic plate count on R2A agar (7-day incubation period at room temperature). The ATP assay showed better correlation with bacterial counts obtained from the 7-day plate count method. This method is also being tested for its usefulness in assessing the water quality following main breaks and flushing operations.

Biological Monitors

A fish biological monitor is presently being installed at the influent to the Hinckley Reservoir Treatment Plant. Using the "avoidance principle," five tanks have been linked together in a series. Native minnows are fed in the upstream tank, which has been designed to be most like their native habitat.

Flow Cytometry

The RBD2100 flow cytometer manufactured by Advanced Analytical Technologies Inc. (AATI) has been used by MVW to detect toxigenic *E. coli*. The procedure allows for identification, in less than four hours, by using fluorescent-tagged antibodies specific to *E. coli* O157:H7. Our laboratory has also been working along with AATI to develop a rapid method using modifications to USEPA method 1623 and the flow cytometer to identify *Cryptosporidium* oocysts. Preliminary studies have been done using reagent water spiked with preserved *Cryptosporidium* oocysts. Recoveries of over 70% have been achieved using this method modification (Schreppel et al., 2002). Further study must be done using matrix waters and live oocysts to assess the feasibility of this method.

CONCLUSIONS

Water system distribution networks are a major area of vulnerability. Water systems need the ability to predict the movement of contaminants and to monitor these levels at points throughout the distribution system. This is an extremely complex process that will require distribution water quality modeling and real-time monitoring. The combination of these two processes will begin to provide an effective tool for enhancing the security of the water system. While it isn't reasonable or feasible to use real-time monitoring for every agent that could be introduced deliberately into a water system, it is practical to monitor using indicator parameters where a change can signal the possibility of hazards in the water. Monitors must be able to be remotely operated, maintainable, not overly sensitive, quality assured, and, last but not least, affordable. Mechanisms must also be in place to interpret the data that the monitoring produces and communications and response actions must be planned. While September 11 2001 taught us that there is no such thing as an "unthinkable act," it has prompted utilities to better monitor distribution systems for water quality, which in turn will benefit our everyday lives. Peace of mind? Better water quality monitoring may bring us one step closer but will we ever really have peace of mind again?

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CHAPTER · 31

Automated Pathogen Detection Systems for Water Quality Monitoring

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BACKGROUND

There has been a long-standing need to develop autonomous pathogen detection systems for water samples that will eliminate the time, variability, and operator involvement needed to obtain accurate results. Construction and validation of an autonomous system for pathogen detection in water supplies is a significant challenge that must take into account (1) the representative volume of water that must be sampled and examined to have statistical confidence in the results, (2) the methods used for concentrating the sample into reasonable volumes for analyses, (3) the processing and purification steps needed for detection, (4) the detection platform chosen (e.g., cell culture, PCR, fluorescent antibody), and finally (5) the engineering required to accomplish these tasks that will provide precise and accurate results with limited false-positive and false-negative error rates. The authors are currently testing a breadboard field device that was originally designed for aerosolized bioterrorism agents. However, the guiding biochemistry that drives the system can be readily adapted for water quality monitoring. The prototype field device is currently being developed for *E. coli* O157:H7. The device can autonomously sample up to 100 mL volumes of water. Processing to remove inhibitors is accomplished by flow-through immunomagnetic separation. The purified cells, immobilized on the magnetic beads, are then moved to a flow-through PCR system, and PCR is performed directly off of the beads. The eluate is collected and the PCR sample is hybridized to an array with specific probes for the detection of these products. Both specificity and sensitivity of the automated protocol are excellent. Consistent recovery of 10 spiked cells into matrix water

(Columbia River water) is routinely achieved. The system architecture and PCR thermal cycling protocols virtually eliminate PCR carryover, allowing the system to be used many times before tubing and other easy user-replaceable parts are required. The next-generation system will be able to collect much larger volumes of water (up to 10 L) and link highly multiplexed PCR for multiple agents, with a flow-through bead-array system for true autonomous detection of any waterborne pathogen.

INTRODUCTION

Available Early Warning Monitors for Pathogen Detection

Most of the recent work for early warning monitors for autonomous pathogen detection has had a military emphasis, and has focused strictly on air sampling methods. Lawrence Livermore National Laboratory has developed and is currently field-testing their Autonomous Pathogen Detection System (APDS). Their system is currently the size of an ATM, and is a complete system that samples large (1,000 to 10,000 L) air samples. Analytes of interest are fluidized and mixed with color-coded polystyrene beads containing specific antibodies (each color bead population is assigned its own antibody) for select agent bacteria, viruses, and toxins. Similar integrated systems and handheld field-deployable units for aerosol monitoring are under development at several other US Department of Energy and Department of Defense Laboratories, but relatively few details of these systems exist in the open literature.

Retrofitting these systems for multi-agent pathogen detection in raw and treated drinking water for an early warning monitor system will present a formidable challenge, regardless of the detection platform chosen (e.g., culture-based detection, protein, nucleic acid, etc.). In their review of candidate methods applicable to drinking water, Straub and Chandler [27] summarized some of the technical challenges for constructing such a system:

1. Very few methods exist for the simultaneous concentration of bacteria, viruses, and protozoan parasites from large (≥ 10 L) volumes of water [14, 17, 18, 21, 22]. These large volumes are needed based on current regulatory practices and the relatively low concentrations of pathogens in these samples. For turbid

water samples, secondary concentration and sample processing were required to render the sample suitable for exam by cell culture or nucleic acid–based methods. Furthermore, even though these samples could co-collect bacteria, viruses, and protozoan parasites, separate assays were needed for each virus or organism group [20].

2. New-generation micro-fluidic and flow-through systems for further sample concentration and processing are in the development and testing phases, but in general, can only handle small sample volumes (in the nanoliter to microliter [μL] range) on rapid time scales desired for detection [2, 5, 10, 13, 23–25, 29, 30].
3. New-generation nucleic acid methods such as DNA microarray technology offer promise for highly multiplexed organism and gene detection, but further developments are needed to realize the goal of true multiplexing [1, 6, 11, 12, 15, 16, 19, 26, 28, 31].
4. Integrating the disparate but connected processes of large-volume filtration, purification, and detection will require not only engineering solutions, but also a thorough understanding of the integrated biochemistry needed to deliver a system that meets current regulatory and emergency response needs.

Of the technical challenges listed above, the concentration and purification step *is the limiting technology* to deliver a product suitable for examination by more traditional cell culture, biochemical, and microscopic methods, as well as new-generation nucleic acid methods. To realize the goal of true, multiplexed detection of bacteria, viruses, and protozoan parasites, any new method should be capable of efficient co-concentration of these pathogen groups. New-generation microparticle-based technologies may be amenable to adaptation for concentration and purification of large-volume water samples to volumes needed for most field-based detectors. This is the approach Battelle has been developing for the analysis of complex environmental samples including air, soil, water, and food.

BIODETECTION ENABLING ANALYTE DELIVERY SYSTEMS (BEADS)

Theory Behind the Biodetection Enabling Analyte Delivery System (BEADS) Platform

From the literature reviewed, derivatized microbeads form the basis of many new-generation nucleic acid and cell purification methods for separating organisms of interest from their interfering environmental matrices. Commercial products like DNEasy (or RNEasy), BIO101, and MoBio utilize chaotropic salts or phenol:chloroform to lyse whole cells or viral particles. Silica-packed microcolumns are then used to capture the nucleic acids. Additional wash solutions are applied to remove interfering matrix. The nucleic acids are then released from the columns in a form that is PCR ready. Likewise, magnetic beads derivatized with specific antibodies can be used to capture and purify specific cells (e.g., *E. coli* O157:H7, *Cryptosporidium parvum*, and *Giardia lamblia*, to name a few) from their surrounding environment. The primary disadvantage of these "kit" methods is that most are batch capture protocols and are not conducive to high-throughput or automated screening.

The authors realized that these same interactive microbead surfaces that have been successfully deployed in batch protocols could form the backbone of an automated flow-through system. The key to the development of this system was the renewable surface column (Figure 1). Fresh, fluidized microbeads are delivered and packed in a column that has a leaky tolerance to allow soluble materials to exit the column. Samples are then perfused over the column and the analytes of interest are captured on the column. Wash solutions are applied to the column to flush unwanted matrix. Additional wash solutions can then be applied, freeing the analyte of interest for further downstream manipulation and analysis, or they can remain associated with the microbead surface. In the latter case, the microbeads are moved to the next fluidics module in the system, where PCR or direct fluorescent antibody detection can be performed. In both cases (free analyte or microbead-associated analyte), the renewable surface column is now empty. Short, automated cleaning processes ensure that the renewable surface column is ready to process another sample. This system eliminates sample carryover that could bias the results of subsequent samples.

The basic concept of the renewable surface flow through columns forms the central theme of Battelle's BEADS (Biodetection Enabling Analyte Delivery System) platform. The platform itself is the combination of automated sample collection and processing through these

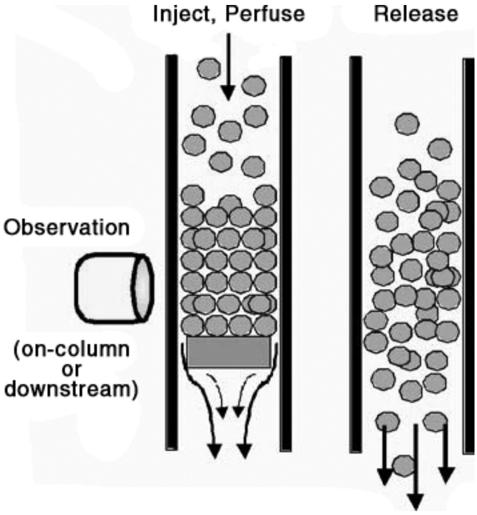

Figure 1 Renewable separation columns function by automatically capturing small volumes (typically 1 microliter) of particles from a slurry. The particles are then perfused with a sample, and analyte(s) of interest bind onto the interactive surface of the particles. The analyte is then either detected on the beads or eluted and detected downstream from the beads. After analysis, the beads are automatically flushed from the column. When applied to biosensing, we call this approach BEADS, for Biodetection Enabling Analyte Delivery System.

columns combined with automated downstream biodetection. We have developed and tested several different types of columns designed to handle different types of microbead particles useful for analytical separations (Figure 2). Materials for these microcolumns include, but are not limited to, silica gel resins for purification of community DNA [4, 8], immunomagnetic beads for capturing whole cells of interest [7], hydroxyapatite, and oligonucleotides tethered to microspheres for affinity purification [9]. Some examples of these separation techniques for whole-cell and nucleic acid capture and analysis are further discussed.

Preliminary Work

Battelle's work in automated DNA sample preparation began with the purification of DNA from soil and sediment extracts [9]. Soil is a very challenging sample matrix since organic soil components such as humic acids will inhibit DNA amplification (PCR), and the presence of other soil components (including proteins and DNA) can interfere with

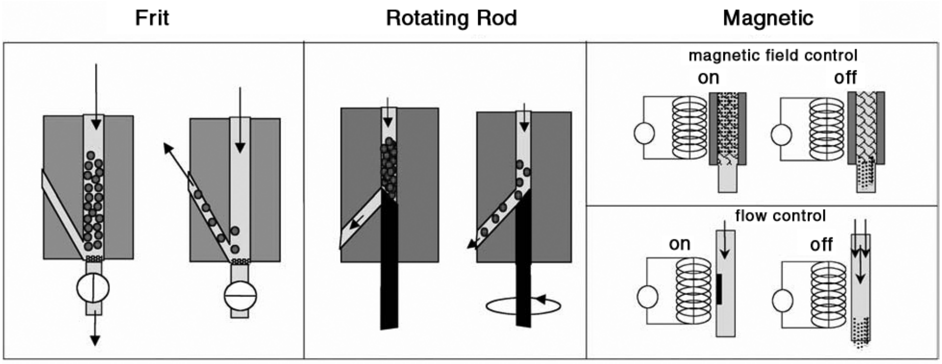

Figure 2 Schematic of some of the Battelle BEADS renewable surface column flow-through cells for automated sample preparation. The type of column used in an integrated BEADS system depends on the user's needs. A frit column and rotating rod column are suitable for handling nonmagnetic particles ranging from 5 to 150 microns in diameter. These particles are usually silica, glass, derivatized polymers, or hydrogels. Two magnetic flow cell approaches have been developed for manipulating magnetic microbeads: an approach that captures beads on a nickel foam throughout the flow path by controlling the magnetic field, and an approach that captures beads on the inside wall of the tubing using a constant magnetic field but variable flow rate to capture and release the beads.

detection. Consequently, scientists at Battelle evaluated several different bead types for DNA purification from soil. Using a frit separation column, they purified DNA from sediments onto 60-micron-diameter polyacrylamide beads derivatized with a universal 16S rDNA capture oligonucleotide. They also used the Battelle magnetic flow cell in combination with superparamagnetic particles derivatized with the same 16S rDNA capture oligo to purify bacterial DNA from a sediment sample. A rotating rod renewable separation column was also used for purifying DNA from sediment, but in this case, they used a total DNA binding reagent that consists of irregular silica particles [8]. In all cases, the DNA eluted from the columns was immediately suitable for PCR. In addition, detection limits were similar to or better than batch capture experiments with the same reagents [3, 8, 9].

Current Applications

The rotating rod column is currently being integrated into Livermore's APDS system for aerosol monitoring. In this application, the rotating rod serves as an antibody affinity matrix. The renewable surface column is first packed with larger-diameter BANGS beads, which serve as a support for antibody-derivatized Luminex fluorescent carboxylated microspheres (Figure 3). Luminex beads are then packed, and samples are perfused over the rotating rod column. After appropriate washes, the microbeads are flushed into the Luminex detector and read. Initial results show that the rotating rod column works quite well: the samples were free from interfering debris, suspected detects could be immediately confirmed by PCR, the larger BANGS beads did not interfere with the optical readout, it was reproducible with no sample carryover, and it could meet the demands of 24/7 aerosol monitoring.

Integrated Immunomagnetic Cell Capture and Detection

The authors have recently assembled an integrated DNA sample preparation and detection system that is designed for unattended monitoring in the field. A liquid sample ranging from microliters to hundreds of milliliters in size is aspirated into the system, and bacterial cells are concentrated onto magnetic beads using the Battelle magnetic flow cell system developed in previous studies [8, 9]. The magnetic beads are then moved to the DNA amplification module for cell lysis (during heating) and DNA amplification using PCR. Finally, the amplified DNA can be

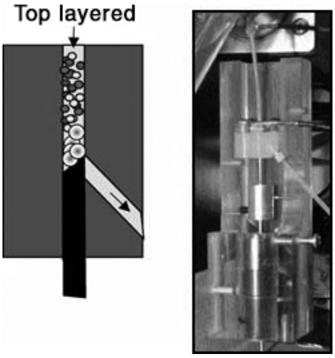

Figure 3 BEADS rotating rod column integrated into Livermore's APDS system. Larger BANGS beads serve as a support for the color-coded antibody-coated Luminex beads (left schematic). The complete column module (right picture) is about the size of a glass microscope slide (25 mm × 75 mm) with an internal diameter of 1–2 mm and packed bed volume of 1–5 μL.

fluorescently detected within the integrated system or off-line by hybridization to a microarray. The fluidics and electronics are separated into two portable boxes, with the fluidics box being about 25 cm on each side (Figure 4).

The authors have successfully sampled up to 100 mL volumes of poultry rinsate through this system [7] with a detection limit of 50 cells/ 100 mL. For raw water, suspended solids in 1-L volumes of Columbia River water (about 10 ntu) were concentrated to a 1-mL slurry. These concentrates were spiked with 10, 100, and 1,000 cells of *E. coli* O157:H7. Triplicate experiments at each concentration were performed. Between each spike run, blank samples were run to verify cleaning procedures between runs. PCR products were recovered and hybridized to a microarray for 1 hr, and then visualized (Figure 5). The key results of these experiments were:

1. Sampling, capture, and PCR were performed in approximately *2 hr.*
2. There was *no operator involvement* once the sample was started.

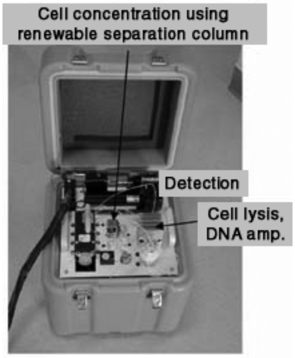

Figure 4 Integrated flow-through system using Battelle's magnetic flow-through cell as the renewable separation column. To date, the largest sample volume that has been tested in this system is 100 mL, but relatively easy to scale up to liter volumes. Samples concentrated and purified on immunomagnetic beads are then moved to a peltier device where cell lysis and DNA amplification by PCR occur. Onboard detection is possible, but the authors are currently working on linking this system with a Luminex flow cytometer for readout of multiplexed PCR products.

3. *Reproducible 10 cell* detection from turbid matrix water (about 10 ntu).
4. *No sample carryover*. PCR was performed using dUTP and UDG to prevent carryover. In addition, rigorous cleaning protocols provided a system that was ready to process the next sample.
5. The *entire process* from sample to readout on the array could be completed in 4 hours.

Given the success of this system, the authors are currently working on multiplexing the immunobead capture to target more organisms of interest. This will be one of the first tasks for development of the next-generation system. Concomitantly, the authors are developing highly

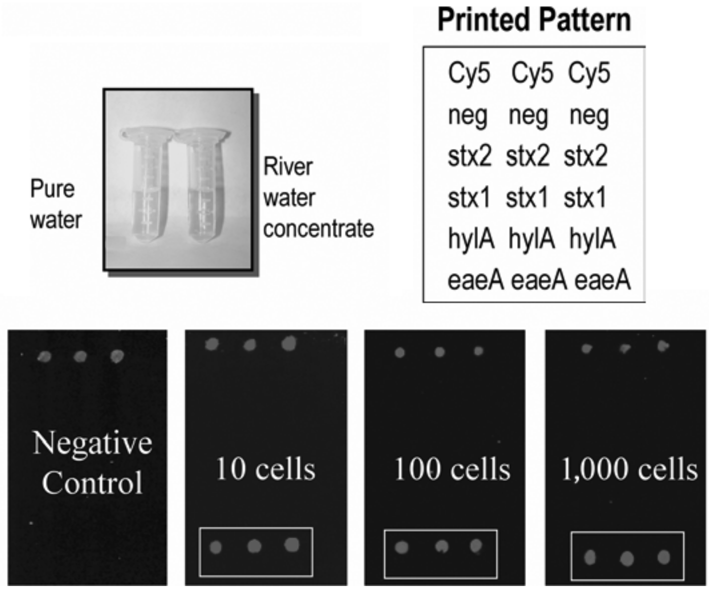

Figure 5 BEADS biobox results for one replicate concentration run. The fluorescently labeled PCR product from the system (20 μL) was manually transferred to a microarray for hybridization and detection. The three hybridized spots on the bottom of the microarray are triplicate probes targeting the eaeA gene of *E. coli* O157:H7. The three spots on the upper side of each image are positive control markers for positional reference to the array. Three other genes are represented on this array, but are not hybridized because the PCR reaction contained only the primer pair specific for the eaeA gene.

multiplexed PCR experiments and hybridizing the products to a bead-suspension array (Figure 6). The goal is to use the suspension array as the automated detector component of the BEADS magnetic flow-through system.

CONCLUSIONS AND FUTURE DIRECTIONS

The authors believe that an integrated system using the magnetic flow-through cell as the renewable surface column provides the best chance for developing a unified waterborne pathogen detection protocol that will ultimately (1) sample and concentrate large volumes of water (up to 10 L) to volumes immediately suitable for downstream analysis; (2) process this

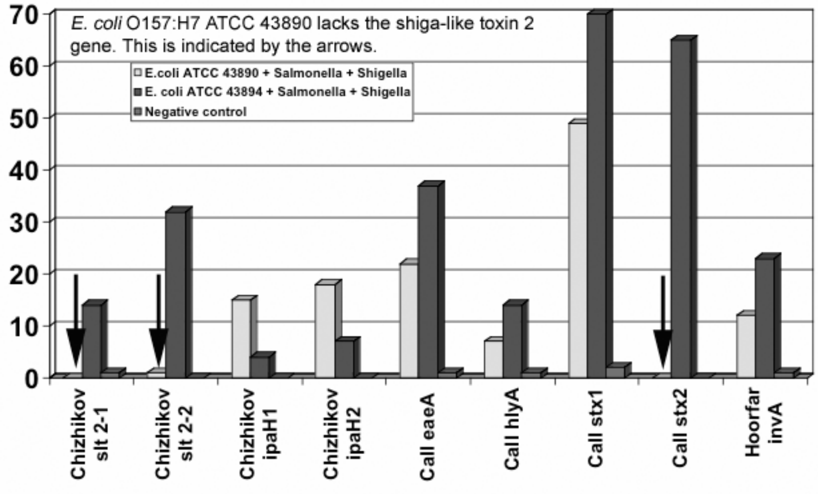

Figure 6 Suspension array results for six different genes representing three different organisms. A six-plex PCR was performed using 100 cells each of *E. coli* O157:H7, *Salmonella* spp., and *Shigella flexneri*. In the first experiment, *E. coli* O157:H7 ATCC 43890 was used. This strain lacks its shiga-like toxin 2 gene (Chizhikov slt2-1, slt2-2, and Call stx2), and the three arrows indicate this. In the second experiment, *E. coli* O157:H7 ATCC 43894 was used. This strain has both shiga-like toxin genes. *Salmonella* was specifically detected using the *invA* gene, and *Shigella* was detected using the *ipaH* gene. Positive controls were significantly above-background, negative PCR controls. Suspension arrays can be easily integrated into the BEADS biobox for completely automated pathogen detection.

concentrate to remove inhibitors; (3) perform multiplexed PCR targeting bacteria, viruses, and protozoan parasites; and (4) detect using a bead suspension array. These operations will be performed autonomously with little to no operator involvement and require little technical training. The versatility of this system will allow "on-the-fly" addition of affinity reagents that will easily allow more agents of interest to be detected, rather than extensive development and validation of a new assay system for a newly emerging contaminant.

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CHAPTER · 32

Evaluation of the DeltaTox and Eclox Systems for Water Toxicity Screening

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SUMMARY

The possibility of chemical and biological activity has become more imminent since September 11, 2001. An attack or threat toward various public places, monuments, businesses, and public utilities could yield detrimental results leading to massive loss of life. This alone has raised the consciousness of many drinking water utilities, prompting them to take more precautionary measures. American Water has taken preventive steps to possibly initiate water monitoring of their systems in the event of possible chemical contamination. The DeltaTox instrument from Strategic Diagnostics and the Eclox instrument from Severn Trent Services are used to provide rapid test methods for screening toxicity of environmental (water) samples.

For this study, the DeltaTox and Eclox instruments were evaluated to assess their capabilities. The first area is to establish a water system (Belleville Lab tap water) baseline, determine their sensitivities to various treatment chemicals and contaminants, and determine the effects of various water quality parameters on their systems.

BACKGROUND

DeltaTox

The DeltaTox system is a rapid test method for screening the toxicity of environmental samples. The system's methodology is based upon the utilization of a highly sensitive analyzer (luminometer) and freeze-dried luminescent bacteria (*Vibrio fischeri*). The luminometer measures light produced by luminescent bacteria after exposure to a test sample and compares the light output to a control. The measured percent light loss or gain is indicative of biological inhibition or stimulation. Light loss results from the metabolic inhibition of the test organism that is proportioned to the relative toxicity of the sample. Light gain indicates a stimulation of the organism by the sample.

Eclox (Enhanced Chemiluminescence Oxidation Reaction)

Unlike the DeltaTox system, which uses live bacteria, the Eclox system is based upon an enzymatic reaction. The Eclox system derives its name from its mode of action (enhanced chemiluminescence oxidation reaction), in which a chemiluminescence reaction of luminol and an oxidant in the presence of an enzyme, horseradish peroxidase, results in the production of light. The light output is measured via a luminometer as percent inhibition of the enzymatic reaction. Many contaminants, such as phenols, amines, and heavy metals, will attack the enzyme itself and reduce the light output [3].

Reaction: Enzyme + Luminol + Oxidant + Catalyst + Enhancer → Light glow

In addition to the chemiluminescence reaction, the Eclox kit provides individual assay materials to detect pesticides, nerve agents, mustard gas, and arsenic. It also contains test systems to determine total dissolved solids (TDS), pH, chlorine content, and color.

RESULTS

I. Comparison of the DeltaTox and Eclox Using Belleville Lab Tap Water

Tap water was analyzed using the DeltaTox and Eclox over a 22-day period to establish baseline values for distributed water. Variability ranged from an average inhibition of 14.5% to –16% (the enzyme can be affected to produce more light). During this time the pH ranged from 7.3 to 7.4, the TDS was a constant 430 mg/L, and the free chlorine content was 0.2 mg/L.

The DeltaTox system showed an average light loss of 44%. Water samples were treated with sodium thiosulfate to neutralize the chlorine residual prior to DeltaTox analysis. The results fluctuated between 28% and 53% inhibition and showed no clear relationship with the Eclox results. It is assumed that both test results reflect normal variability when applied to tap water with no particular significance in the patterns.

II. Impact of Commonly Used Water Treatment Chemicals on the DeltaTox and Eclox Systems

Various water treatment chemicals were studied using the DeltaTox and Eclox systems at the following concentrations to determine their effect. An average of the two replicates analyzed is presented in Table 1.

The following treatment chemicals were examined to determine their impact on the DeltaTox and Eclox systems: zinc chloride, ammonium chloride, aluminum, chlorine, copper sulfate, and manganese. The DeltaTox system showed increased sensitivity to low levels of zinc chloride, ammonium chloride, and copper sulfate. Light loss, greater than 50%, was seen with the DeltaTox system but showed little impact with the Eclox system.

The Eclox reaction was not substantially impacted by zinc chloride, ferric chloride, and fluoride or aluminum chloride. Ammonium chloride showed 20% inhibition at 8 mg/L. Copper sulfate at 2 mg/L showed 40% inhibition, and at 10 mg/L showed 79% inhibition.

Manganese sulfate showed increasing inhibition with increasing concentrations in the Eclox system but did not affect the DeltaTox system. These studies showed that the Eclox system was sensitive to low concentrations of manganese, resulting in a 95% inhibition at 0.5 mg/L. Manganese occurs naturally in many surface and groundwater systems, and approximately 40% of 193 American Water treatment facilities had source water manganese levels >0.05 mg/L and 25% of 112 groundwater facilities had finished water manganese levels >0.01 mg/L [5].

Chlorine at 1.0 mg/L and 5.0 mg/L increased inhibition in both systems. Chlorine at 1.0 mg/L showed a 69% inhibition and 100% inhibition at 5 mg/L in the Eclox system. The DeltaTox system was greatly impacted by both concentrations of chlorine (98–100% light loss). Without neutralization of chlorine with 1% sodium thiosulfate, at 1.0 mg/L or 5.0 mg/L complete light loss was observed in the DeltaTox system. An insignificant effect (9%, 16% light loss) was seen upon neutralization with sodium thiosulfate.

Neutralization with sodium thiosulfate of the 1.0 mg/L sample analyzed via the Eclox system actually increased inhibition from 69% to 100%. The enzyme in the Eclox system reacts negatively to thiosulfate. The manufacturer supplies a preconditioner solution that contains an ammonia-based reagent that is used to dechlorinate water samples. After neutralization with the manufacturer-supplied preconditioner solution, a 2–6% inhibition was observed.

Table 1 Treatment chemicals studied

Chemical		DeltaTox % Light Loss (–) or Gain (+)	Eclox % Inhibition of the Enzyme Reaction Activity (+) Stimulated (–) Inhibited
Zinc chloride	0.2 mg/L	–84	–1
	0.5 mg/L	–93	–3
Ammonium chloride	2 mg/L	–42	N/A
	4 mg/L	–52.5	+8
	8 mg/L	–66.5	–19.5
Fluoride	1 mg/L	–9	N/A
	4 mg/L	–8	–9
Ferric chloride	0.2 mg/L	+2.5	N/A
	0.5 mg/L	+4.5	–9.5
Manganese sulfate	0.01mg/L	N/A	–35.5
	0.03 mg/L	N/A	–60.5
	0.05 mg/L	N/A	–73.5
	0.1 mg/L	N/A	–84
	0.5 mg/L	+7	–95
	1 mg/L	+13	–96.5
Chlorine	–1.0 mg/L	–99	–69
	–5.0 mg/L	–100	–100
thiosulfate	+ 1.0 mg/L	–9	–98.5
conditioner	+ 1.0 mg/L	N/A	–10.5
thiosulfate	+ 5.0 mg/L	–16	–99
conditioner	+ 5.0 mg/L	N/A	–6
Copper sulfate	0.01 mg/L	N/A	N/A
	0.2 mg/L	–100	N/A
	0.5 mg/L	–100	+8.5
	1.0 mg/L	N/A	+6
	2.0 mg/L	N/A	–40.5
Aluminum chloride	0.2 mg/L	+22.5	–1
	0.5 mg/L	–44.5	–1

N/A = not analyzed

NOTE: Chemicals analyzed via DeltaTox were prepared in reagent-grade water. Those analyzed via Eclox system were prepared in tap water.

III. Evaluation of the Sensitivity of the DeltaTox and Eclox Systems for Detection of Low-Level Contaminants

Several contaminants were used to evaluate the sensitivity of the DeltaTox and Eclox systems (Table 2). The greatest sensitivity of the DeltaTox system was observed at the higher concentrations of the contaminants. Cyanide at 5 mg/L and 10 mg/L, arsenic at 1.0 mg/L, and lead at 0.5 mg/L showed 100% light loss. However, in a past evaluation [1], when these toxic levels were added to natural water samples, the levels needed to produce inhibition (>90% light loss) tended to be much higher than the levels observed for reagent-grade water. When cyanide was added to water from Missouri American, Peoria, and California, inhibitory (>90% light loss) levels increased to 10 mg/L cyanide [1].

An average of 39.5% light loss was observed at 50 mg/L strychnine. The toxicity associated with strychnine was shown to be primarily attributed to the ethanol used to dissolve the compound. Ethanol alone accounted for 50% to 98% of the toxicity of the highest levels of strychnine [1]. The low solubility of strychnine in water decreases its importance as a waterborne contaminant.

The Eclox test was not particularly sensitive to strychnine, with only 60% inhibition at 50 mg/L. Arsenic at 1.0 mg/L –0.25 mg/L showed 96–100% inhibition, but low levels of sensitivity at concentrations less than 0.063 mg/L. The Eclox test was very sensitive to cyanide, and concentrations greater than 0.002 mg/L showed substantial inhibition (96–100% inhibition). Lead concentrations of 1.0 mg/L–0.25 mg/L showed nearly complete inhibition.

Because the Eclox test was sensitive for detection of cyanide (0.002 mg/L), arsenic (0.125 mg/L), and lead (0.125 mg/L) at very low levels, it can more than likely alert analysts to the presence of these compounds at concentrations below their acute toxicity thresholds [4].

IV. Evaluation of Various Water Quality Parameters

Water samples with various parameters were studied using the DeltaTox system. Samples chosen possessed water quality parameters known to consist of metals, iron, manganese, hardness, arsenic, chelated copper algaecides, turbidity, and high pH. With the exception of high pH, none of the waters tested demonstrated a significant (>25%) impact on the DeltaTox system [1].

Waters with pH 9.5 showed 77.5% average light loss with the DeltaTox. Water samples with pH values greater than 8, or less than 6, will have to be adjusted. It may be possible for the vendor (SDI) to

Table 2 Contaminant study

Contaminant Concentration (mg/L)		DeltaTox % Light Loss (–) or Gain (+)	Eclox Inhibition of the Enzyme Reaction Activity (+) Stimulated (–) Inhibited
Strychnine	1	–4	–5
	5	–2.5	–11.5
	10	–3.5	–21
	50	–39.5	–60.5
Cyanide	0.002	N/A	–95.5
	0.004	N/A	–97.5
	0.008	N/A	–97
	0.016	N/A	–99.5
	0.031	N/A	–99.5
	0.063	N/A	–97.5
	0.125	N/A	–99.5
	0.25	N/A	–100
	0.5	N/A	–100
	1.0	–62.5	–100
	5.0	–100	N/A
	10.0	–100	N/A
Arsenic	0.002	N/A	–6
	0.004	N/A	–11
	0.008	N/A	–13
	0.016	N/A	–18.5
	0.031	N/A	–32.5
	0.050	+5	N/A
	0.063	N/A	–62.5
	0.100	+2	N/A
	0.125	N/A	–93
	0.200	+7	N/A
	0.250	N/A	–96.5
	0.500	+13	–98
	1.0	–100	–96.5

N/A = not analyzed

NOTE: Chemicals analyzed via DeltaTox were prepared in reagent-grade water except strychnine. Strychnine was dissolved in 1.5% EtOH. Those analyzed via Eclox were prepared in tap water.

Table continued next page.

Table 2 Contaminant study *(continued)*

Contaminant	Concentration (mg/L)	DeltaTox % Light Loss (–) or Gain (+)	Eclox Inhibition of the Enzyme Reaction Activity (+) Stimulated (–) Inhibited
Lead	0.002	N/A	–5.5
	0.004	N/A	–7.5
	0.008	N/A	–16
	0.016	N/A	–34.5
	0.031	N/A	–37.5
	0.050	N/A	N/A
	0.063	N/A	–75
	0.100	–31	N/A
	0.200	–79	N/A
	0.250	N/A	–99
	0.500	–90	–99.5
	1.0	N/A	–99

N/A = not analyzed

NOTE: Chemicals analyzed via DeltaTox were prepared in reagent-grade water except strychnine. Strychnine was dissolved in 1.5% EtOH. Those analyzed via Eclox were prepared in tap water.

prepare the reconstitution solution with a buffer that would eliminate the need for pH adjustment in the field.

The pH levels of three tap water samples were adjusted to a pH of 5.4, 9.5, and 7.2 and examined via the Eclox. Tap water at pH 5.4 showed a 5% inhibition. The water sample with a pH of 9.5 showed 3% inhibition. The sample of pH 7.2 showed a 17% inhibition. Varying pH levels had no significant impact on the Eclox system because the reagents used in the chemiluminescence reaction contain buffers to ensure optimum pH for the test [2, 3].

DISCUSSION

These preliminary studies of the DeltaTox and Eclox systems highlight a number of important issues that must be considered in the application of either system for monitoring of waterborne toxicants:

1. Normal levels of water treatment chemicals may have an impact on the system and could produce a false-positive signal under certain circumstances. It is important to adjust the pH of the water sample to 6–8 and neutralize any chlorine residual with the DeltaTox system. Figure 1 shows a 44% light inhibition even with thiosulfate dechlorination of chloraminated water.
2. It will be necessary for water facilities to determine a baseline result for their water system and evaluate deviations from the baseline levels. Additional monitoring is necessary to determine baseline levels for a representative number of systems. Control charts could show useful information when detecting any deviations from the norm. This testing should strive to collect sufficient background data (20–40 measurements) to permit analysis of significant deviations from baseline levels.
3. Contaminants in tap water samples may be less toxic to the DeltaTox system than when tested in reagent-grade water. Additional testing of toxic compounds in representative tap water samples is necessary to determine the sensitivity of the DeltaTox system for contaminants of greatest concern.

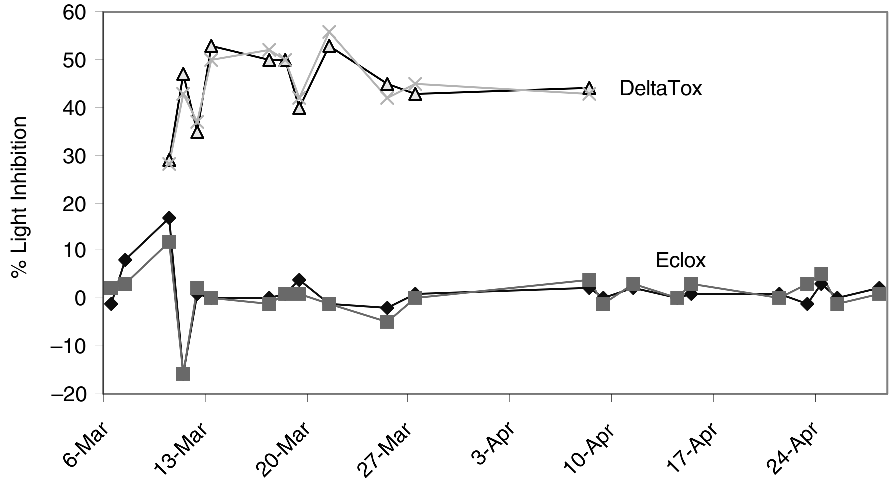

Figure 1 Comparison of the DeltaTox and Eclox using Belleville Lab tap water

Both systems appear to be a relatively simple screening tool for toxic compounds in water samples. The basis of the Eclox assay is similar to that of the DeltaTox system, with the exception that an enzymatic reaction is performed instead of whole living cells. One unique aspect of the Eclox assay is that toxic compounds may affect the enzyme by either increasing or decreasing its activity.

Both systems were sensitive to waters that contained free chlorine and chlorine. Water samples must be neutralized with sodium thiosulfate (DeltaTox); the manufacturer supplies ammonia-based reagent with the Eclox kit. It is possible that some inhibition may be observed in sulfur-containing groundwaters, but this effect was not examined in this trial.

Both the Eclox system and the DeltaTox system were sensitive to common treatment chemicals, but the sensitivity to particular chemicals varied (Table 3). For example, the DeltaTox system showed significant inhibition (>50%) to zinc chloride, ammonium chloride, and copper, whereas these chemicals showed little effect on the Eclox system. The DeltaTox system was sensitive to pH values greater than 8 or less than 6 and must be neutralized with sodium thiosulfate. Unlike the Eclox test, buffers are already present in the reaction reagents upon analysis.

The DeltaTox system was less sensitive to cyanide, and required cyanide levels of 1.0 mg/L to produce an average inhibition of 62%. In contrast, the Eclox system detected cyanide at levels as low as 0.002 mg/L. Similar results for cyanide were reported by States et al. [6]. Similarly, the Eclox system was more sensitive to arsenic than the DeltaTox test. The Eclox system showed an average 93% inhibition when exposed to 0.125 mg/L arsenic, whereas the DeltaTox system showed no sensitivity to arsenic at low levels. Sensitivity was seen with arsenic levels of 1 mg/L. For lead, the Eclox test exhibited a 98% toxicity response at 0.125 mg/L, whereas the DeltaTox system exhibited 79% inhibition at 0.2 mg/L. Neither test was particularly sensitive to strychnine, requiring the highest level (50 mg/L) to observe any substantial effect. The DeltaTox system showed 39.5% light loss, whereas the Eclox system showed 60.5% inhibition at the 50 mg/L concentration.

Cost Study

The cost of the DeltaTox is slightly lower than that of the Eclox kit, and the cost of the DeltaTox reagents is lower than on a per-use basis (Table 4). However, pricing arrangements are available from the manufacturers and discounts are applied to volume purchases.

Table 3 Summary of chemical and contaminant detection limits

Chemicals/Contaminants	DeltaTox	Eclox
Zinc chloride	0.2 mg/L	ND
Ammonium chloride	4 mg/L	ND
Manganese sulfate	ND	0.03 mg/L
Chlorine	1.0 mg/L	1.0 mg/L
Copper sulfate	0.2 mg/L	ND
Cyanide	1.0 mg/L	0.002 mg/L
Arsenic	1.0 mg/L	0.063 mg/L
Lead	0.2 mg/L	0.063 mg/L
Strychnine	ND	50.0 mg/L

Chemicals and contaminants at the various concentrations show > 50% light loss or inhibition. ND = non-detect.

Table 4 Price comparison for DeltaTox and Eclox kits

Item	Price
Strategic Diagnostics Inc.	
DeltaTox Analyzer	$5,900.00
Analyzer + Reagents (150 reactions)	$6,990.00
Microtox Acue reagents (1,000 reactions)	$505.00
Severn Trent Services	
Eclox Analyzer + (consumables for 50 reactions)	$7,900.00
Consumables (kit reagents, 50 reactions)	$695.00

CONCLUSIONS

The DeltaTox and Eclox systems are useful instruments that offer monitoring capabilities. Both the DeltaTox system and the Eclox kit can produce false-positive toxicity results in response to conventional water treatment chemicals, and therefore the kits alone cannot be used to determine the risk of a particular water sample. However, when integrated into a comprehensive monitoring strategy along with disinfectant residual monitoring, pH, conductivity, and visual and aesthetic characteristics, the systems can provide valuable additional information to judge the security of a water supply.

For monitoring of baseline (no contamination) water samples, both the DeltaTox and Eclox systems gave similar trend results, but the DeltaTox showed higher levels of inhibition, presumably due to the residual ammonia in the dechloraminated samples. This may pose a problem for the use of the DeltaTox technology in the water industry.

The inclusion of buffering agents in the Eclox reaction reagents eliminated the need for the operator to adjust the pH when it was greater than 8 or less than 6. Additionally, the Eclox system was more sensitive to levels of arsenic, cyanide, and lead than the DeltaTox assay. Neither system was particularly responsive to low levels of strychnine.

Insufficient data are available to make a recommendation between the Eclox system and the DeltaTox system with respect to performance in the field, customer support, or reliability. Rather, it is concluded that both systems would be useful adjuncts to the routine monitoring of source and finished water quality.

ACKNOWLEDGMENTS

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CHAPTER · 33

Utilization of a New Toxicity Testing System as a Drinking Water Surveillance Tool

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SUMMARY

The wide variety of possible threat agents or contaminants that could be introduced into the nation's drinking water supplies preclude testing for every possible agent. This forces us to rely on screening methods capable of detecting a diversity of analytes in a single test. Toxicity assays are one possibility. What is needed is a low-cost, easy-to-use system that can be widely deployed. The Hach ToxTrak™ method makes use of resazurin reduction to measure respiration, a critical pathway for cell viability. Resazurin is a redox-active dye that, when reduced, changes color from blue to pink. Substances that are toxic to bacteria can inhibit their metabolism and thus inhibit the rate of resazurin reduction. This inhibition of resazurin reduction is taken as an indication of toxicity in the test. The method makes use of an accelerant (gluteraldehyde: US patent 5,413,916) in the method to reduce the reaction time, thus preventing oxygen interference and also allowing the use of a lower level of inoculum to reduce the dye. The use of gluteraldehyde allows measurement on a low-cost instrument such as a spectrophotometer or colorimeter. Even lower-cost visual measurement with a color wheel has been shown to work. Any type of bacterial culture may be used.

This method has been evaluated and found to be effective in detecting contamination by a wide variety of threat agents in drinking water, including heavy metals, herbicides, pesticides, rodenticides, radionuclides, industrial chemicals including cyanide, commercial products, biotoxins, and chemical warfare agent surrogates. The method was found to be effective in detecting almost all compounds tested at levels of interest to human health. This system represents a cost-effective and wide-scope sentinel method for use by drinking water utilities. The low

cost, wide range of effectiveness, and ease of use entailed in this system make it ideal for the development of citizen-based water monitoring groups that could be used as an early warning system in the event of a contamination incident. The method could also find use by first responders and in forensic investigations in tracing contaminant plumes or verifying system incursions.

INTRODUCTION

The events of September 11 have brought the potential vulnerability of critical US infrastructure to the nation's attention. A prime target for terrorist assault is the nation's drinking water systems. An attack aimed at drinking water distribution systems could result in numerous casualties and cripple the population's general psyche. Bogus news reports from a radio show reporting that the local water supply had been contaminated with dihydrogen monoxide (water) caused a near panic in one midwestern community [1]. That water systems are considered a viable target by al-Qaeda and other terrorist organizations has been amply demonstrated by the arrest of a group of men in Rome, Italy, in February 2002. The suspected terrorists were found to be in possession of potassium ferricyanide and maps of the water distribution system around the US embassy [2].

Because of these vulnerabilities drinking water utilities, public health professionals, and first responders have voiced interest in the need for an inexpensive, easy-to-use, quick method to evaluate drinking water suspected of contamination. Already, drinking water professionals are instituting more elaborate and comprehensive laboratory analysis when ratepayers notify utilities about water quality concerns. Public inquiries about water quality are no longer managed as routine consumer complaints. Increased surveillance of finished drinking water increases this need for quickly deployable and easy-to-use test methods. This chapter explores a possible method to detect tampering with the water supplies using an inexpensive test kit system.

TOXICITY TESTING AS A WATER SURVEILLANCE TOOL

The vast number of chemical agents that could be utilized by a terrorist to compromise a water supply system precludes monitoring on an individual chemical basis. Chemical warfare agents such as VX, Sarin, and Soman; commercially available herbicides, pesticides, and rodenticides;

street drugs such as LSD and heroin; heavy metals; radionuclides; cyanide; and a host of other industrial chemicals could be exploited as weapons [3]. This suggests utilizing some general form of toxicity testing to monitor for changes in water integrity [4].

Toxicity is the ability of a chemical or mixture of chemicals to cause a living organism to undergo adverse effects upon exposure. These effects can include negative impacts on survival, growth, and reproduction. Toxicity tests are analytical experiments that attempt to detect or quantify toxicity in a sample by measuring the results exposure produces on standard test organisms.

Toxicity testing has a long history of use. The food testers employed by Roman emperors and medieval kings were a rudimentary form of toxicity testing. The probability of getting someone to volunteer for this sort of work today is not high, and even if volunteers were available, OSHA would most likely object. This presents a problem with toxicity testing for threats to human health. No other organism will respond to a toxin in exactly the same way as a human.

The closer an organism is to humans on the evolutionary tree, the closer its responses to toxins should mirror human responses. This is why clinical toxicologists have long used other mammals, such as monkeys, dogs, rats, and mice, in their toxicological studies.

The expense and slow response time involved with using these types of organisms to monitor water supplies are prohibitive for general applications. The next best alternative would be to use lower-order vertebrates, such as fish, as surrogates. On-line monitoring of captive populations of fish, such as trout, has been utilized for several years, most notably in Europe, to evaluate source water for contamination before it enters treatment plants [5]. This is an expensive and time-consuming arrangement. While technologically simple, the maintenance of fish stocks and the interpretation of changes in fish action or viability can be difficult. Interactions with parameters such as water temperature and turbidity can skew results. Also, the chlorine present after treatment would damage the fish's gill structure and would make such a system useless for monitoring in the distribution system. This type of system has limited applications and would be of most use in larger systems for monitoring surface water feed supplies.

Another option is to use invertebrates. These organisms are even further removed in their equivalence to human responses, but their ease of use and lower maintenance costs make them a viable choice. Instruments using organisms such as the water flea, *Daphnia*, have been developed and are in use [6]. They tend to suffer from some of the same problems as fish-based systems, and the instruments themselves tend to be expensive.

This leaves microorganisms, such as bacteria, as the last viable alternative. While they are far removed from humans and would be expected to exhibit different responses from humans, their simplicity, low cost, and ease of use make them the organisms of choice.

BACTERIAL MONITORING OF TOXICITY

The concept of using microorganisms to monitor toxicity has been used for many years in the wastewater industry as a means to safeguard the bacterial populations in treatment systems from shutdown due to toxic exposure. A wide variety of testing procedures and methods have been developed to monitor the toxic characteristics of influent to treatment plants. One type of commercially available test functions by measuring the effect of toxins on the light output of luminescent bacteria. This test often correlates well with total toxicity, but it does have some limitations. Bioluminescence is not an essential metabolic pathway, nor is it widespread among living organisms. Toxins that specifically inhibit luciferase, the enzyme responsible for bioluminescence, may not exhibit general toxicity to other organisms. In addition, the measurement of bacterial luminescence requires the use of very expensive instrumentation.

Another type of commercially available test measures the cleavage of X-Gal or ONPG by the β-galactosidase enzyme of a strain of *Escherichia coli*, which is highly sensitive to toxins. What is actually being measured in this test is the ability of one specific strain of bacteria to synthesize the enzyme β-galactosidase. The test is limited to one genetically engineered strain of *Escherichia coli* that may or may not be representative of general toxicity.

A third type of test is also commercially available. It measures the reduction of oxygen consumption by bacteria. Oxygen consumption is a good measurement of overall bacterial health, but this method does present some problems. Drawbacks are that the test requires the use of a dissolved oxygen electrode, an expensive piece of equipment. The samples must be aerated for 30 min before starting the test. This aeration could result in the loss of volatile toxins from the sample. Finally, only one test can be run at a time; multiple tests require long periods of time coupled with extensive sample manipulation.

The above tests are designed to be used with a specific organism. Users are not given the option of determining the toxicity of substances to other microbes or one indigenous to their own system. The inability of the other methods to utilize the normal flora contained in a given system, along with cost per test, is a major concern that is addressed by the proposed method described here.

CHEMISTRY OF THE PROPOSED METHOD EXPLAINED

Liu et al., in an article entitled "A Rapid Biochemical Test for Measuring Chemical Toxicity" (*Bulletin of Environmental Contamination and Toxicology*, 26:140–149 [1981]), described a biochemical toxicity test based on the reduction of resazurin by bacterial respiration. Resazurin is a redox-active dye that, when reduced, changes color from blue to pink. Resazurin is in the oxidized, blue, state at the beginning of the test. The bacteria oxidize the glucose added to the sample with the dye and reduce the resazurin (see photo). First, the resazurin is reduced by two electrons to resorufin, which has a pink color. Resorufin can be further reduced by two more electrons to dihydroresorufin, which is colorless. Dihydroresorufin can be reoxidized by atmospheric oxygen to resorufin. It is important that readings be taken before a significant amount of resorufin has been reduced to prevent interference. If the reaction time is too long, the indicator is too far reduced and interference will result. Substances that are toxic to bacteria can inhibit their metabolism and thus inhibit the rate of resazurin reduction. This inhibition of resazurin reduction is

Blanks on right turn from blue to red during the course of the test while the samples on the right remain blue due to inhibition of oxidation (color image on page 1083).

taken as an indication of toxicity in the test. The Liu method involves an organic solvent extraction and centrifugation; these steps are time-consuming and cumbersome. Also, the extracting solvent used, n-amyl alcohol, has toxic properties that may affect test results.

The method described in this chapter follows the same concept as Liu's method but has been modified to alleviate some of its drawbacks. The proposed method makes use of an accelerant (gluteraldehyde: US patent 5,413,916 assigned to Hach Co.) to reduce the reaction time. This prevents oxygen interference and also allows the use of a lower level of inoculum to reduce the dye. Therefore, due to the decrease in turbidity resulting from a smaller inoculum, the absorbance of the dye can be read in a colorimeter or spectrophotometer without removing the bacterial cells from the light path. While the exact mode of action that allows gluteraldehyde to speed up the process of respiration is unknown, it is most likely the result of the uncoupling of oxidation from phosphorylation during respiration.

Respiration is the process that generates ATP (adenosine triphosphate) using the energy from the oxidation of an electron donor by an external electron acceptor. In the case of aerobic respiration the terminal electron acceptor is oxygen (O_2). Electrons are transferred from the electron donor to the terminal electron acceptor (resazurin in the test) through a series of electron-transfer proteins that are embedded in the cell membrane. Substances such as redox reactive dyes like resazurin can act as alternative electron acceptors by oxidizing one of the membrane proteins.

Electron transfer is coupled to ATP generation by a proton gradient across the cell membrane that is created as electrons flow through the membrane-bound electron transfer system. ATP is synthesized from ADP (adenosine diphosphate) and inorganic phosphate when this proton gradient is used to drive the phosphorylation of ADP by a membrane spanning ATPase.

Since it involves the creation and maintenance of a proton gradient across the cell membrane, respiration is affected by substances that make the membrane permeable to protons. When protons can cross the membrane freely, bacteria cannot use a proton gradient to make ATP. The oxidation of energy sources is thereby uncoupled from the phosphorylation of ADP. Substances that uncouple oxidation from phosphorylation in this way are often organic acids like phenols that are lipid soluble but can become protonated on the outside of the membrane, diffuse across the membrane, and then become deprotonated on the inside of the membrane.

Since the rate of a catabolic process is controlled by the ADP/ATP ratio, the uncoupling of oxidation from phosphorylation accelerates these processes. Some substances that cannot act as proton shuttles uncouple oxidation from phosphorylation by inhibiting the ATPase that uses the proton gradient to synthesize ATP. It is possible that gluteraldehyde acts in this way.

Gluteraldehyde has the ability to accelerate the resazurin reaction in many different species of bacteria, including both Gram-positive and Gram-negative species. There is a possibility that it will work with some eukaryotic species, though this has not been tested. This allows the user to culture his/her own organisms and determine the toxicity of a sample to the bacteria that will be exposed to it. Regardless of the mechanism, the result of the action of uncouplers (possibly gluteraldehyde) in the toxicity test is twofold. Reactions that are dependent upon the concentration of reduced enzymes and ATP are slowed due to a decrease in the concentration of substrates. On the other hand, reactions like the reduction of resazurin that are dependent upon the rate of electron transfer are accelerated.

The use of gluteraldehyde to accelerate the reaction allows the avoidance of many of the problems associated with Liu's original method. It allows the use of a smaller number of bacteria to reduce the resazurin dye. Consequently, the absorbance of the dye can be read instrumentally without removing the bacterial cells from the light path. This alleviates the need for organic extraction and/or centrifugation. With the reduced turbidity, the color change of the dye is dramatic and can be easily distinguished visually. Also, the decreased reaction time eliminates the interference caused by overreduction of the dye. Thus, the proposed test can be performed rapidly compared with other tests. In addition, the proposed method allows the utilization of an inexpensive colorimetric technique without extraction, filtering, or centrifuging. The test can be used as a visual method. Finally, this test, unlike other tests, can be used with many different species of bacteria or mixed cultures.

PROCEDURE

As with all toxicity tests using surrogate organisms, it is important to remember that this method is not all-inclusive. There are substances that are detrimental to humans that will show no response to this test. There are also substances, such as copper, that may show a dramatic response with this method and yet represent little or no human toxicity.

1. First, the inoculum for the test must be prepared. This is done by growing a bacterial culture at 35°C in a lauryl tryptose broth tube for 10 to 72 hr. The culture should be visibly turbid. The bacteria can be obtained from the system being monitored, or standard stock cultures may be used as a source of the inoculum.
2. Next, determine the number of tests to be run; several samples from different sources or serial dilution of a single source may be run at the same time. At least one blank needs to be run at the same time as the sample(s). This blank can consist of DI water that exhibits no measurable toxicity, or another water source that has been chosen to represent zero or baseline toxicity may be used. For the monitoring of drinking water samples, it may be best to obtain a large reservoir of water that will be retained and used as a baseline from which any deviation can be charted. This will cancel out most superfluous contaminants, such as copper, that are not of interest. Sample and blank water samples should be at the same temperature. After the number of tests to be run has been determined, add 5 mL of blank water or sample to the appropriate tubes.
3. Add 1 drop of 0.0246 *N* sodium thiosulfate to each of the tubes, including the blank, to remove any residual chlorine. This step may be omitted if the water is not chlorinated.
4. Add 2 drops of the accelerator solution (gluteraldehyde) to each tube.
5. Then add one of the reagent pillows. These pillows contain the resazurin dye and all of the food and nutrient sources needed to support active cell respiration.
6. Add 0.5 mL of the culture prepared in step 1 to each of the tubes. The cultures should be agitated before use to ensure even dispersal of the organisms.
7. Shake vigorously to dissolve all of the powder and mix the solution. Shaking also serves to fully oxygenate the samples, ensuring that oxygen concentration is not a factor in determining respiration rate.
8. Measure and record the Abs for each tube at 603 nm.

9. Allow the solutions in the tubes to react until the Abs of the blank has decreased 0.600 ± 0.100. This should take 45–75 min. (Exact reaction time will vary depending upon a number of factors, including temperature, age of culture, and number of bacteria per 0.5 mL. Note that incubation of the tubes should occur at room temperature. The accelerator solution will speed the reaction at room temperature [20–25°C], but it will tend to decrease the rate of reaction at 35°C.)
10. After the blank has decreased in Abs 0.600 ± 0.100, measure and record the Abs for each tube.
11. Calculate the change in Abs for each tube.
    $\Delta\text{Abs} = \text{Abs step 8} - \text{Abs step 10}$
12. Calculate the percent inhibition for each sample.
    $\%\ I = \{1 - (\Delta\text{Abs sample} \div \Delta\text{Abs blank})\} \times 100\%$

INTERPRETING RESULTS

The percent inhibition results obtained are only a relative measurement and do not represent a true quantitative measurement of toxic concentration. The percent inhibition does not necessarily increase in direct proportion to the concentration of toxins. In order to determine the minimum inhibition concentration of a toxin, it is possible to make tenfold dilutions of the sample and determine the percent inhibition for the dilutions until the sample is diluted sufficiently so that no inhibition is observed. This is known as the no observed effect concentration (NOEC).

Due to the many variables involved in the test, the limits of detection are on the order of 10% inhibition. This would correlate to the lowest observed effect concentration (LOEC). If a sample shows less than 10% Inhibition, the test should be repeated. After several repetitions it is advisable to look at the series of data to determine the likelihood of toxicity. Results below 10% are not reliable, but can be used to surmise some presence of toxicity if they are consistent. See examples in Table 1.

Some toxins will increase respiration and will give a negative percent inhibition on this and all other respiration-based toxicity tests. After repeated testing, samples that always give a percent inhibition that is more negative than –10% should be considered toxic. Some compounds may exhibit a change from negative to positive inhibition or vice versa as the concentration of the substance is increased. This may be due

Table 1 Percent inhibition results

Data Points: % Inhibition	Conclusion
7%, 9%, 5%, 8%, 5%	May be slightly toxic.
7%, –4%, –5%, 5%, 1%	Most likely not toxic.
–7%, –9%, –5%, –8%, –5%	May be slightly toxic.

to the differing effects of the substance. For example, at low concentrations it may act as a supplemental food source or nutrient, while at higher levels it acts as a toxin.

EXPERIMENTAL RESULTS—BASELINE STUDY

The first step in establishing a monitoring program for a given tap water system is to establish a baseline on water of acceptable quality. All tests can be run versus DI water as a baseline, but to center the results on zero toxicity, it may be preferable to establish a reservoir of the water to be tested and run all tests versus that source. This will compensate for any metals or other toxicants that are routinely present in the water to be monitored.

As an example, a baseline was established for the tap water at the Hach facility in Loveland, Colorado. The day before the series was to begin, the tap water was allowed to run for 10 minutes to equilibrate and then a 5-L fluorinated, high-density polyethylene container was filled with the water to use as a set point to run all future tests against.

For the course of the study, the tap water was allowed to run for 10 minutes and then a sample was taken each day. The proposed procedure was performed using the water from the reservoir as the blank. The water temperature was allowed to equilibrate before testing. A stock culture of *E. coli* derived from Aqua-QC Stiks™ was used as the inoculant. (Note: Any lyophilized bacteria culture will suffice. Aqua-QC Stiks™ is a registered trademark of Microbiologics Inc.) Thiosulfate was added to both the samples and the blanks to ensure uniformity. All tests were done in triplicate and an average of the results was reported (see Figure 1).

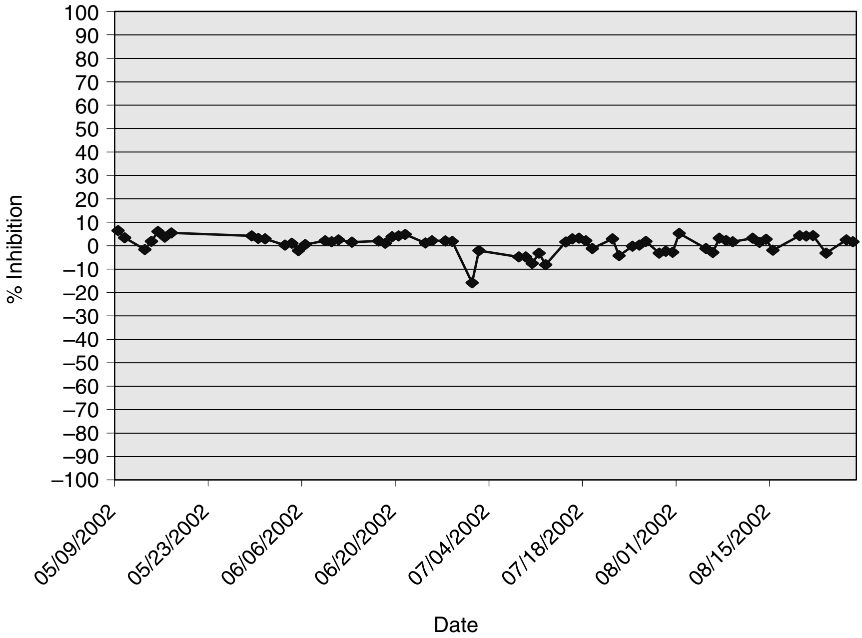


Figure 1 Baseline of tap water toxicity (Loveland, Colo.)

INVESTIGATION OF COMPOUNDS OF CONCERN FOR DRINKING WATER SECURITY

There are a large number of compounds that could be deployed by terrorists in an assault on water supplies. Among these are heavy metals; commercially available herbicides, pesticides, and rodenticides; radionuclides; industrial chemicals including cyanide; and chemical warfare agents such as VX, Sarin, and Soman.

No form of toxicity test can be expected to respond to all of these agents. In the following experiments, a number of these compounds were investigated as to their ability to elicit a response from the proposed test.

The agents being tested were dissolved in DI water and tested versus the same DI water source without agent added. All tests were run in triplicate, and the number reported is the average response. If it was necessary to add a solvent such as methanol to get a given reagent to dissolve, a like amount was added to the DI water blank so as to ensure that the response was from the agent and not due to solvent effects.

Heavy Metals

Heavy metals are agents of concern due to their toxicity to humans. They are also fairly easy to obtain, and their salts tend to be readily soluble. A number of metals were investigated at various concentrations (see Table 2).

Herbicides Investigated

While as a general class, herbicides tend to be less detrimental to human health than some other compounds, there are some notable exceptions (e.g., Endothal) [7]. This, along with the ability to easily obtain large quantities of these chemicals from agricultural supply sources, adds to the concern. Even if few fatalities resulted, the panic caused by the introduction of herbicide-type compounds into a water system could be severe; therefore, a number of likely compounds were investigated (see Table 3).

Insecticides Investigated

Insecticides tend to be more harmful to human health than herbicides. Some of the insecticides have chemical structures quite similar to some of the chemical warfare nerve agents, and there are several that are cholinesterase inhibitors [7]. Like herbicides, insecticides are also readily available in large quantities. For some, their solubility limits their usefulness as water-introduced weapons, but others are quite soluble and

Table 2 Response of heavy metals

Compound	Concentration	% Inhibition
Arsenic	1.0 ppm	34.0
Arsenic	0.1 ppm	42.0
Cadmium	10.0 ppm	–28.0
Cadmium	1.0 ppm	–25.4
Lead	10.0 ppm	–26.5
Lead	1.0 ppm	–10.1
Mercury	10.0 ppm	48.0
Mercury	1.0 ppm	31.0
Mercury	0.1 ppm	10.0
Thallium	5.0 ppm	–27.4
Thallium	1.0 ppm	–14.5

Table 3 Response of herbicides

Compound	Concentration	% Inhibition
Alachlor	1.0 ppm	12.0
Cacodylic acid	5.0 ppm	–23.2
Cacodylic acid	1.0 ppm	–10.7
Cyanazine	10.0 ppm	–10.1
Cyanazine	1.0 ppm	–15.4
Diquat	100.0 ppm	–37.3
Diquat	10.0 ppm	–24.2
Diquat	1.0 ppm	–21.4
Endothal	1.0 ppm	–34.0
Endothal	0.1 ppm	–13.0
Glyphosate	2.0 ppm	15.0
Paraquat	10.0 ppm	–18.1
Paraquat	1.0 ppm	–10.7

present more of a threat. A number of these compounds were investigated (see Table 4).

Nematocides and Rodenticides Investigated

Nematocides are similar to insecticides. With some exceptions, they do tend to be more soluble than insecticides. Some nematocide compounds are also similar to chemical warfare agents in structure and mode of action. Rodenticides are of concern because they are specifically designed to be lethal to mammalian species such as humans. Both classes are readily available in large quantities. A number of these compounds were investigated (see Table 5).

Industrial Chemicals and Miscellaneous Agents

There are any number of industrial chemicals that could be used in an attack. Chief among these is cyanide, which is widely used in mining and other industries. That terrorists are thinking along these lines is borne out by the fact that the attempted terrorist act in Italy involved a cyanide salt [2]. While the compound in question was of limited toxicity, the mere fact that it had cyanide in its name probably led the terrorists to

Table 4 Response of insecticides

Compound	Concentration	% Inhibition
Azinphosmethyl	1.0 ppm	–21.0
Carbaryl	20.0 ppm	No response
Carbofuran	10.0 ppm	28.1
Carbofuran	1.0 ppm	12.2
Dichlorvos	100.0 ppm	–21.1
Dichlorvos	10.0 ppm	–13.0
Lindane	5.0 ppm	22.0
Malathion	2.0 ppm	–17.5
Malathion	5.0 ppm	–20.1
Methomyl	1.0 ppm	–27.8
Methomyl	10.0 ppm	12.4
Methyl Parathion	50.0 ppm	21.0
Methyl parathion	10.0 ppm	14.0
Methyl Parathion	1.0 ppm	10.2
Nicotine	5.0 ppm	14.5
Phorate	10.0 ppm	19.1
Phorate	1.0 ppm	11.4
Turbufos	10.0 ppm	30.0
Turbufos	1.0 ppm	24.4

Table 5 Response of nematocides and rodenticides

Compound	Concentration	% Inhibition
Bromodiolon	5.0 ppm	–12.1
Fenamiphos	10.0 ppm	–18.7
Fenamiphos	1.0 ppm	–14.2
Fluoracetate	10.0 ppm	–28.4
Fluoracetate	1.0 ppm	–12.2
Oxamyl	1.0 ppm	–16.4
Strychnine	100.0 ppm	–28.5
Strychnine	25.0 ppm	–27.2
Strychnine	10.0 ppm	–19.1
Strychnine	5.0 ppm	–15.2

attempt to use it. Other industrial and miscellaneous chemicals could also be used. Several were investigated (see Table 6).

Radionuclides

The use of radionuclides as a terror weapon is a distinct possibility. Even if casualties were low, the psychological aspect of a nuclear threat could be severe. Obtaining high-purity, highly radioactive material, such as plutonium or uranium 238, is difficult, and it is unlikely that a terrorist organization that had obtained these materials would be inclined to use them in an attack on a water system. More likely is the use of low-level radioactive material or waste. Low-level radioactive salts of uranium and thorium were studied (see Table 7).

Table 6 Response of industrial and miscellaneous agents

Compound	Concentration	% Inhibition
Cyanide	10.0 ppm	28.0
Cyanide	1.0 ppm	21.0
Cyanide	0.1 ppm	10.0
Fluoride	10.0 ppm	–21.2
Fluoride	1.0 ppm	–53.8
Sodium Diethyl Dithiocarbamate	10.0 ppm	–35.0
Sodium Diethyl Dithiocarbamate	1.0 ppm	–12.0
PCB (Arochlor 1248)	20 ppb	16.5

Table 7 Response of radionuclides

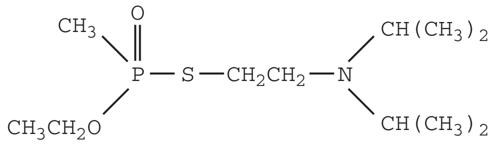
Compound	Concentration	% Inhibition
Uranium	10.0 ppm	–24.6
Uranium	1.0 ppm	–22.8
Uranium	0.1 ppm	–18.7
Uranium	0.01 ppm	–11.2
Thorium	10.0 ppm	–18.8
Thorium	1.0 ppm	–11.8
Thorium	0.1 ppm	–10.1

Commercial Products

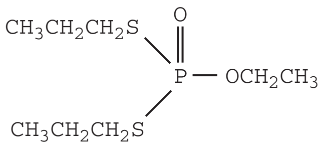
Though they are not the weapons of choice for organized terrorists, lone saboteurs, the emotionally unstable, or small groups may turn to easily obtained commercial products such as bug sprays or lawn chemicals. Many of the active ingredients of these preparations are the same as in the pesticides and herbicides already discussed. The difference lies in the smaller proportion of active ingredients. The vast majority of these compounds have inert ingredients listed as their main component. Also, many of these compounds contain a mixture of different active ingredients. These factors may lead to a different toxicity profile from that of the pure ingredients. A few are evaluated in Table 8.

Chemical Warfare Agents

Chemical warfare agents such as VX, Soman, and T-2 toxin, along with older-type chemical weapons such as mustard gas and Lewisite, are not likely to be targeted against a water system. It is more likely that any assault from these weapons will be via aerosol. As the result of an aerosol attack, it is possible and even likely that these agents could find their way into water supplies. Most warfare agents were not available for testing. Aflatoxin was tested along with Ethoprophos, a surrogate for VX nerve agent (see Figure 2 and Table 9).



VX Nerve Agent



Ethoprophos

Figure 2 Chemical structure of VX nerve agent and Ethoprophos

Table 8 Response of commercial products

Compound	Concentration	% Inhibition
Roundup[*]	100 ppm	12.8
Roundup[*]	10 ppm	–5.4
Ortho Wasp & Hornet Spray[†]	1,000 ppm	13
Ortho Wasp & Hornet Spray[†]	100 ppm	11
Ortho Wasp & Hornet Spray[†]	10 ppm	–20.1
RealKill Multipurpose Lawn & Garden Insect Killer (Dursban)[‡]	1,000 ppm	22.9
RealKill Multipurpose Lawn & Garden Insect Killer (Dursban)[‡]	100 ppm	9.3
RealKill Multipurpose Lawn & Garden Insect Killer (Dursban)[‡]	10 ppm	–28
Hot Shot Spider Killer[§]	1,000 ppm	8.7
Hot Shot Spider Killer[§]	100 ppm	5.0
Hot Shot Spider Killer[§]	10 ppm	–36.7
Triacide (Diazinon)[**]	1,000 ppm	–13.4
Triacide (Diazinon)[**]	100 ppm	–17.7
Triacide (Diazinon)[**]	10 ppm	–43.2

[*] Roundup is 50.2% Glyphosate and 49.8% inert ingredients.

[†] Ortho Wasp & Hornet Spray is 0.2% Tetramethrin, 0.2% Sumithrin, and 99.6% inert ingredients.

[‡] RealKill Multipurpose Lawn and Garden Insect Killer or Dursban is 2.5% Permithrin and 97.5% inert ingredients.

[§] Hot Shot Spider Killer is 0.03% Tralomethrin, 0.05% d-Trans Allethrin, and 99.92% inert ingredients.

[**] Triacide or Diazinon is 0.5% Lambda-Cyhalothin and 99.5% inert ingredients.

DISCUSSION OF RESULTS

The purpose of running a baseline study was to determine if the baseline would hold steady enough to allow recognition of significant events in the water system. As a whole, the test performed very well in the baseline study. Except for one anomalous data point, the range held between 6.4 and –8.1% inhibition. This is well within the ± 10% expected for the test.

Table 9 Response of warfare agents

Compound	Concentration	% Inhibition
β Aflatoxin	10 ppm	11.0
β Aflatoxin	1 ppm	–27.0
β Aflatoxin	0.1 ppm	–28.3
Ethoprophos	1 ppm	–23.0
Ethoprophos	0.1 ppm	–11.0
Ethoprophos	0.01 ppm	–10.0

A reformulation of the reagent powder is undergoing study and will hopefully cut down on this variability even further.

There was one anomalous data point (<–15% inhibition). This appeared to be some sort of significant change. As there were no reports of illness in the area, it can be concluded that it was most likely not a serious contamination event.

A possible explanation for the sudden change in toxicity could be the massive forest fires in the area at the time. Throughout most of the monitoring period, the area was hazy from the smoke, but on the day in question the blanket of smoke was extremely heavy. In fact, it appeared as a moderate fog and the odor could be distinctly detected. This was the only day during the monitoring period that the smoke was that heavy and may offer an explanation for the spike in toxicity.

An analysis of possible agents that a terrorist organization could use was done, and a list was compiled. As many of these reagents as possible were obtained and analyzed for their response to the proposed toxicity test. The vast majority of the reagents showed a response to the test. Some of the heavy metals exhibited a positive effect on the toxicity test (mercury and arsenic) while the others (cadmium, lead, and thallium) showed negative toxicity. Both positive (respiration has sped up) and negative (respiration has slowed) results greater than 10% or less than –10% are considered toxic. All of the levels tested showed a response greater than the noise in the test at levels far below what would be expected to cause acute human toxicity. The test appears to be a very good indicator of contamination by heavy metals.

It must be kept in mind, however, that false positives are possible. Copper, at levels as low as 2 mg/L, has been shown to exhibit toxicity levels in the test as high as 70%. That is why it is important to run the test versus a baseline reservoir that contains levels of metals commonly found in the system.

The herbicides that were tested, as was the case with the heavy metals, exhibited a wide range of responses, with both positive and negative inhibition. The test was able to detect all of the herbicides screened at the stated levels. One result that appears strange is that Cyanazine exhibits a greater response at lower doses than it does at higher doses. This discrepancy is within the noise level of the test, but may also be explained if, at higher levels, the bacteria were able to begin using the compound as a carbon source, which could help to counteract some of its toxic effects during the short time frame of the test. Another factor worth noting is that the response to Endothal, which is one of the herbicides with the largest known toxic effects, occurred at very low levels and was up to –34% at only 1 ppm.

The insecticides also showed a variety of responses, with both positive and negative inhibition being exhibited. Positive inhibition seemed to be more prevalent. The test showed a response to all of the insecticides investigated except for Carbaryl. Carbaryl is fairly difficult to dissolve in water, making it an unlikely candidate for use as a weapon against water supplies. To get it into solution it was necessary to place it in an ultrasonic bath for several hours. The heat generated in this process may have resulted in chemical changes rendering it less toxic. On the other hand, *E. coli* bacteria may not respond to this toxin and the non-response of the test may be valid. There are definitely some substances that will not elicit a response from this test, whether Carbaryl is one of them or not.

Another interesting response found in the insecticide data is exhibited by Methomyl. This compound exhibited strong negative inhibition at –27.8% at a concentration of 1 mg/L and then switched to positive inhibition of 12.4% when the concentration increased to 10 mg/L. This may be due to competing modes of toxicity or to the use of the compound by the bacteria at a low concentration as a carbon source, which is overwhelmed by its ability to slow respiration as the concentration increases. There should theoretically be a point between 1 and 10 mg/L when the toxicity is equal to zero. The chances of an event hitting the exact concentration whereby the test would be unable to detect it are slim.

All of the nematocides and rodenticides investigated gave a response to the test. The responses for this class all showed a negative inhibition. Common poisons like strychnine and fluoracetate were capable of being detected at levels as low as 1 mg/L.

The industrial and miscellaneous chemicals also all showed a response. The responses exhibited both positive and negative inhibition. Cyanide, an industrial chemical of prime concern due to its availability, toxicity, and easy solubility, was detectable down to 0.1 mg/L. Another response of note was that of fluoride. It had a large response at as little as 1 mg/L. Fluoride is commonly added to drinking water. This helps to show the importance of using a reservoir for the baseline.

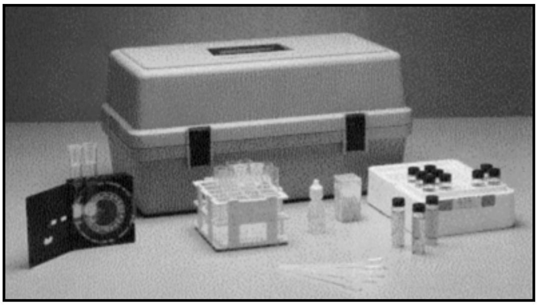
Radionuclides are not readily available; therefore the compounds tested were radioactive but not extremely so. All of the compounds tested exhibited negative inhibition. It is not certain whether or not the exhibited toxicity was due to the radioactive nature of the materials or due to their action as heavy metals. The test, however, did show good sensitivity to both thorium and uranium. Thorium was detectable down to 0.1 mg/L and uranium responded down to 0.01 mg/L.

The commercial products tested gave mixed results. All showed up as toxic at some level. The responses were complicated due to the fact that the majority of these compounds consist of inert ingredients. These inert ingredients are not necessarily inert to the test. Also, some of the products have more than one active ingredient that may exhibit competing modes of toxicity. One component may be driving the inhibition negative while the other drives it positive. The inert ingredient may also act as a toxicant or as a food source, further complicating the response. Regardless of this, all of the compounds tested showed some response, either > 10% or < –10% at some concentration level.

The only warfare agent available for testing was β Aflatoxin. It showed a strong negative inhibition at low concentrations but drifted to positive at higher concentrations. This is probably due to its use as a food source. While nerve agents were not available, a surrogate, Ethoprophos, was tested (see Figure 2). The results showed that this toxin could be detected at levels as low as 0.01 ppm. Similar results could be expected for VX nerve agent.

CONCLUSIONS

The new toxicity-monitoring tool described in this chapter has been shown to be effective at detecting a wide variety of possible chemical threat agents. The test has been shown to be capable of exhibiting a response from each category of agent tested, and all agents except one, Carbaryl, showed a response at one concentration or another. The test is extremely effective at detecting very low doses of some contaminants, such as heavy metals, radionuclides, and cyanide.



The ToxTrak™ Toxicity Testing Kit

ToxTrak™ being used under first responder conditions

The fact that the test can be run on any spectrophotometer or colorimeter capable of measuring absorbance at around 600 nm means that most facilities will have no startup instrumentation costs when adopting this method. Most water treatment facilities already possess an instrument of this type. Luminescent-based systems would require the purchase of a luminometer, which is an expensive instrument. The ability to use the test on a portable colorimeter or with a color matching system, such as a color disk, combined with the rapid time to results, makes the system useful for first responders doing spot checks of water quality. The procedure is not complicated and can be easily mastered by users with a minimum of technical experience. Reagent costs are minimal at around $2 per test.

An additional advantage to this method, especially for small cash-strapped municipalities, is that it can be used for more than one purpose. The original concept design for this test was as a tool to monitor wastewater influent to prevent disruptions to a wastewater treatment system. This use becomes even more important with the possibility of terrorists striking a waste treatment system to affect drinking water intakes downstream. It can also serve a role as an investigative tool to help track pollution plumes to their source and as an initial monitoring tool to reduce costs when whole effluent toxicity monitoring (WET testing) becomes necessary.

The proposed tests' simplicity, versatility, and low cost for instrumentation compared with other methods of toxicity monitoring should make it a valuable tool in the arsenal of solutions we deploy to help safeguard our nation's drinking water supplies.

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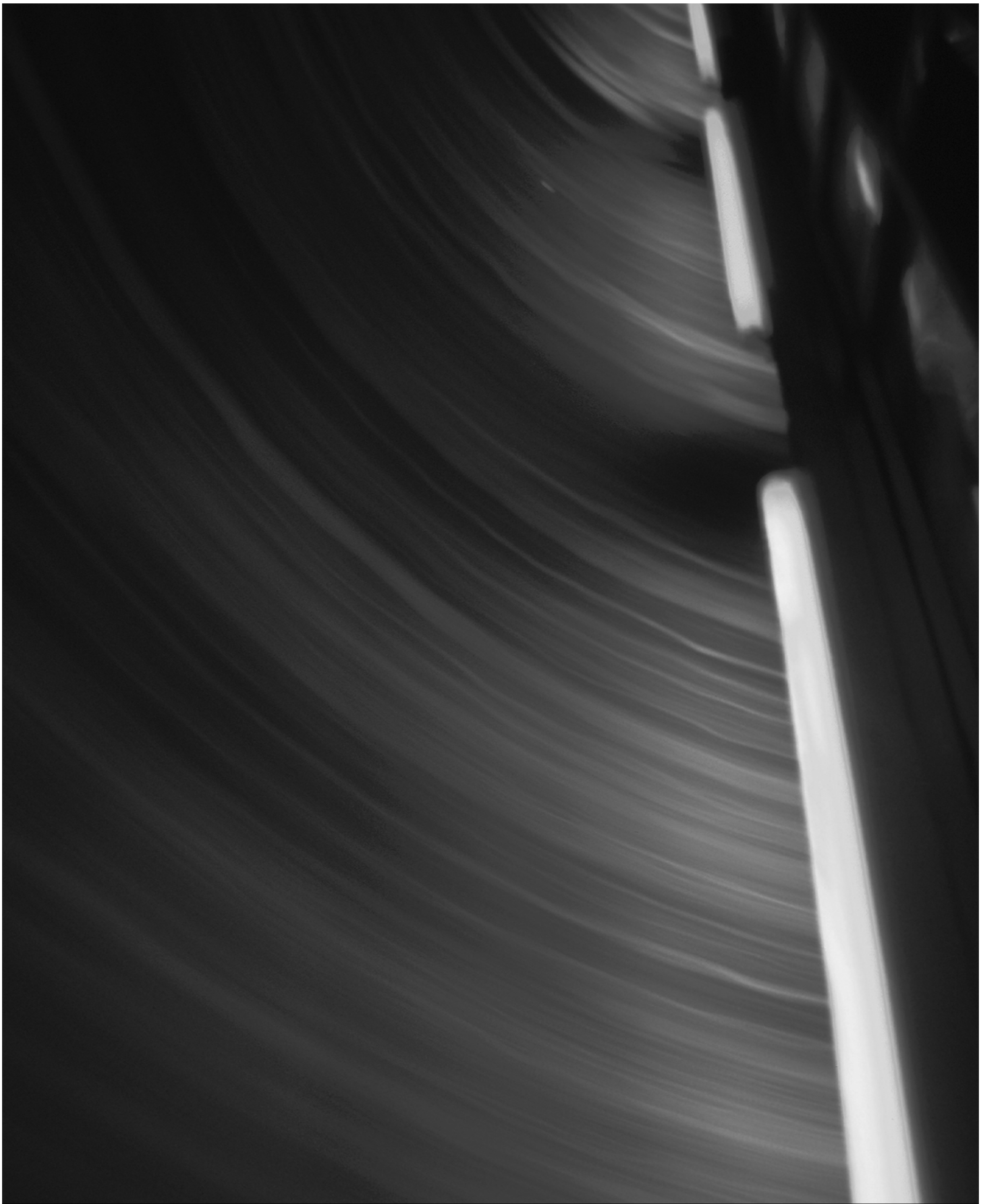

Part 7

Operational Practices

CHAPTER · 34

Improving Distribution System Performance

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BACKGROUND

Customer satisfaction requires adequate distribution system performance in terms of minimal service interruptions, adequate pressure, and adequate water quality. Many small and medium-sized utilities struggle to achieve adequate distribution system performance because they have infrastructure, technical capacity, managerial capacity, or financial capacity limitations. Small and medium-sized utilities can overcome these limitations by employing strategies that focus attention and resources on key system performance benchmarks.

PERFORMANCE BENCHMARKS

The author believes that customer satisfaction is the most important measure of distribution system performance. Customer satisfaction requires that water always be available, that it have sufficient pressure, and that it be of good quality. Experience with several small and medium-sized systems suggests that customers are most concerned with having an uninterrupted supply of water.

Benchmarks related to customer satisfaction are used to gauge distribution system performance. These benchmarks have been selected on an ad hoc basis from regulatory agency requirements (NJAC 7:10 et seq. and NJAC 7:19-6.4), the initiative of a forward-thinking utility (Lehigh County Authority, 2000), published information (AWWA, 1996; Kirmeyer et al., 2000), and information accessible through the Internet (QualServe, 2002). Benchmarks have been selected on an ad hoc basis because utility management had neither an appreciation for nor resources for more formal development of customized benchmarks. Small and medium-sized utility managers, in the author's experience, rarely have either an appreciation for or resources for the development of customized benchmarks because managing the utility is only one of several competing responsibilities, because no improvement in performance can be attributed directly to the effort to identify benchmarks, and because formal benchmarking may require a much larger percentage of the utility's resources than doing so would for a large utility. Nevertheless, using benchmark terminology when communicating with managers has been effective in focusing attention and resources on key issues related to improving system performance.

The benchmarks being used to gauge performance with the upper management of small and medium-sized utilities are:

Benchmark	Issue	Objective	Goal	Origin
Service outages	Reliability	Percent of time customers receive service	Outages < 26 min. per year (99.995% reliability)	LCA, 2000
Water main breaks	Reliability	Breaks per 100 miles of pipe	20	Kirmeyer et al., 2000; LCA, 2000
Pressure	Adequacy	Percent of time pressure greater than 20 pounds per square inch	100	NJAC 7:10-11
Total coliform bacteria	Water Quality	Number of positive total coliform samples	Zero	NJAC 7:10-5; LCA, 2000
Total trihalomethanes	Water Quality	Annual running average total trihalomethane concentration, mg/L	0.1	NJAC 7:10-5
Lead	Water Quality	Concentration of 90th-percentile sample, mg/L	0.015	NJAC 7:10-5
Unaccounted-for water	Efficiency	Percent of water produced or purchased not accounted in sales or other uses	15	LCA, 2000; NJAC 7:10–7:19-6.4
Cost per customer	Efficiency	Cost per retail customer, \$/yr/person	NA	NA

The water industry database (AWWA, 1996) contains information to relate some benchmarks to utility performance nationwide. For example, 50% of all participating utilities reported 12 or more water main breaks per year per 100 miles of pipe; more than 90% of the participating utilities reported no positive total coliforms; more than 90% of the participating utilities reported total trihalomethanes less than 0.069 mg/L; 50% of all participating utilities reported more than 13% unaccounted-for water; and more than 50% of all participating utilities reported costs greater than $83 per retail person served (1991–1995).

These data are for utilities of all sizes throughout the United States. More representative statistics can be extracted from the database should circumstances warrant.

STRATEGIC PLANNING

Managers of small and medium-sized systems in older communities share some common frustrations. Their systems are over 50 years old. The systems have high percentages of undersized mains. The undersized mains are unlined cast-iron or asbestos-cement pipe. Many of these utilities serve communities with limited financial resources available for improvements. Complicating matters are increased expectations, especially in terms of water quality standards. Small and medium-sized utilities frequently are part of municipal governments responsible for satisfying a wide range of community needs, which means that their managers may have a hard time getting upper management's attention until the customers revolt. Having a strategic plan grounded in the community's or utility's organizational goals is one way to manage these frustrations and make the right decisions for the future.

Strategic planning by small or medium-sized systems occurs usually in the context of addressing other planning needs, if at all, in the author's experience. For example, many utilities prepare capital improvement plans to satisfy regulatory agencies. These plans, covering a minimum of five years, have been the principal tool used by the author to introduce strategic planning concepts to the managers of several small and medium-sized utilities in the eastern United States. Concepts have been abstracted from Malcolm Pirnie's strategic planning experience with larger utilities and from recent articles (Shanaghan and Beecher, 2001; Spacek and Matichich, 2002). The Malcolm Pirnie approach focuses on relating performance measures or benchmarks to organization goals, past problems, and "core business practices." Annual capital plan updates and in some instances other regulatory pressures have provided the opportunities to work with utility managers to help them envision the programmatic and infrastructure improvements that will increase the performance of their distribution systems. The two most recent examples of this approach, the Township of Mahwah, N.J., and the Borough of Haledon, N.J., both consider customer satisfaction a very important organizational goal.

The strategies being employed to improve performance target the distribution system's infrastructure and the utility's technical, managerial, and financial capabilities. The managerial capabilities are the key to successfully improving performance because they determine priorities for all the other strategies. Managers need to

- Focus on long-term organizational goals
- Understand why some core business practices have not been followed
- Make staff accountable through the use of benchmarks

Although management is the key, understanding how to improve the utility's infrastructure is a prerequisite to improving the distribution system's performance. The process of planning and implementing is iterative; each planning cycle leads to establishing priorities for both capital improvements and staff activities, which in turn feed into the next year's plan.

INFRASTRUCTURE IMPROVEMENTS

Old pipes, especially unlined cast-iron pipes, are plagued by tuberculation, biofilms, and sediment. These conditions weaken the pipes, making them more prone to breaks, reduce their hydraulic capacity, harbor microbiological contaminants, reduce disinfectant residuals, increase disinfectant by-products, and interfere with corrosion control.

Old distribution systems, in addition to having less adequate pipes, have smaller pipes, inoperative valves, more dead ends, and less redundancy. Smaller pipes frequently lead to inadequate pressure. The lack of pipe loops and inoperative valves multiplies the negative impact of water main breaks and other service interruptions. The dead ends increase the travel time and reduce mixing, increasing water quality problems. Frequently water storage tanks accumulate sediment or mix inadequately, which contributes to water quality problems.

Old or absent supervisory control and data acquisition systems hinder improvements to system control and performance measurement.

Many types of improvements have been made to address each of these three types of infrastructure needs. Spreadsheets based upon how each particular improvement would contribute to achieving the performance benchmarks were used to prioritize infrastructure improvements because limited financial and managerial capabilities preclude replacing entire distribution systems. Improvements that contributed to achieving more than one benchmark were assigned higher priorities.

Strategies used to minimize sediment and biofilms included more careful management of disinfectant residuals, improved flushing programs, and cleaning or replacing mains when flushing alone could not remove deposits.

For example Haledon, N.J., has been practicing supplemental chlorination since 1999 to reduce the frequency of positive coliform tests. Over 50% of the pipe in the system is unlined cast iron. Many of these pipes are severely tuberculated, which both consumes disinfectant residuals and protects microorganisms from its effect. Routine sampling sites had tested positive for total coliforms several times from 1997 through 1999. Supplemental disinfectant residual monitoring has helped identify which portions of the distribution system were most at risk because of low or absent disinfectant residuals. As it turned out, the utility clearly had a problem maintaining the residual in the pressure zone most distant from the system entry point. Haledon began feeding additional chlorine at the entry point to that zone in 1999. Later that year it began feeding additional chlorine at the entry point to the entire system. Since 1999, working with its supplier, the Passaic Valley Water Commission, it has made provisions to convert, using chloramines as the supplemental disinfectant. The capital cost for the initial supplemental feed was under $10,000 and the operating cost has been about $1,000 per year. The system's entry point had equipment to feed chlorine so there was no capital cost, but the operating cost was about $3,000 per year. The capital cost of the chloramination facilities was about $100,000. Haledon has not had a positive coliform test since 2000 due to improved disinfection practices and other improvements.

Flushing programs, especially unidirectional flushing programs, have been effective in removing sediment and biofilms from pipes. Unidirectional flushing is a systematic program of cleaning from larger mains to smaller mains by flushing pipe at velocities greater than five feet per second, always using clean water coming from previously flushed mains. Malcolm Pirnie is a proponent of unidirectional flushing (Joseph and Pimblett, 2000). Haledon's system has such a simple configuration that the flushing program was laid out by the superintendent without using optimizing software. However, more typically, Malcolm Pirnie uses Aqua Data software to optimize the unidirectional flushing program. The other facets of planning a flushing program are ensuring that all necessary valves and hydrants are operable and that all dead ends can be flushed at an adequate velocity. Haledon has invested more effort in its valve maintenance program over the last five years to ensure that the valves are operable. In 2001 it contracted out to provide hydrants or blowoffs as appropriate for the diameter of the existing main at 15 dead ends, for a cost of $60,000. The author believes that the improved water quality of Haledon's distribution system is due at least in part to the improvements made in the flushing program.

Some mains need to be mechanically cleaned to remove tuberculation, sediment, and biofilms. Lining the main with cement after cleaning extends the benefit of cleaning several years. Cleaning and lining mains is increasingly popular. Bloomfield, N.J., began cleaning mains in 1985 in order to reduce red water complaints. Bloomfield has lined about 90 miles of main, or 90% of its distribution system, since then at a cost of approximately $15 million. Red water complaints have been eliminated. Pressure at hydrants on dead ends is much improved, according to the fire department. Haledon, too, has been cleaning and lining key transmission mains to restore hydraulic capacity. Haledon has lined about half a mile, or 3% of its system, at a cost of approximately $250,000 in order to restore the rated capacity of its principal entry point.

Distribution systems need to be reinforced, repaired, and replaced to satisfy reliability and adequacy (pressure) benchmarks.

Operable valves are important for reliability as well as for flushing. The New Jersey Board of Public Utilities mandates annual exercising of major valves and biennual exercising of other valves for the water utilities under its jurisdiction. Documenting the location and condition of valves and an adequate valve maintenance program are important staff performance measurements. The author accepts handwritten records in recognition of the limited computer literacy of many utility operators. For small and medium-sized systems, computerized asset management incorporating valve maintenance is still part of the long-range plan.

Redundancy is the antidote to system vulnerability characterized by the existence of critical components, which, if out of service for more than 24 hours, would result in widespread service interruptions. Vulnerability is very system-specific. The critical component may be a pumping station, a storage tank, or even a main.

Haledon's system is exceptionally vulnerable. Eighty percent of its transmission mains are unparalleled by other mains. It has a single pump station serving 40% of its customers. Unfortunately, the system's vulnerability was demonstrated after Hurricane Floyd in 1999 when about 50% of the utility's customers were without water for days. One of the main objectives of Haledon's capital improvement program has been reducing vulnerability (improving reliability). Two million dollars is being spent for improvements needed to create the necessary redundancy.

Mahwah's system is also vulnerable. A distribution system evaluation (Elam Associates, 1996) identified transmission main improvements totaling about $5 million to create transmission main loops that would provide redundant mains to supply major population concentrations within the Township.

Appropriate pipe diameters contribute to adequate pressure and water quality. Mains that are too small cannot sustain adequate pressure under fire flow conditions. Mains that are too large cannot sustain adequate water quality under average conditions.

Satisfying current firefighting water requirements generally requires at least 8-in. mains under any circumstance where more than one hydrant needs to be supplied or the distance from the nearest junction is more than a couple of hundred feet. Replacing undersized mains, which also are frequently made from less desirable materials like cast iron or even galvanized steel pipe, is a widespread upgrade because it simultaneously improves several performance measurements.

Approximately 25% of the pipe in Haledon's distribution system is smaller than six inches in diameter. Another 40% is six inches in diameter. The first replacement of undersized mains is scheduled to begin this year. Replacing undersized mains has a lower priority than improving redundancy. The highest replacement priorities are mains that have frequent breaks and mains serving areas where the existing undersized main reduces fire protection. It will be several years before all of Haledon's undersized mains are replaced. The plan is to reevaluate replacement vs. cleaning and lining for 6-in. mains as the program progresses. The probable cost of the program is in the range of $5 million to $10 million.

Mahwah has experienced fewer water quality problems than Haledon but had neighborhoods where customers complained of inadequate pressure. Mains up to 75 years old, four inches in diameter or smaller, served these neighborhoods. Mahwah has now replaced almost all mains smaller than six inches in diameter and is starting to replace older cast-iron mains. Dead ends are also being eliminated as part of the program wherever possible.

Haledon has a 16-in. main that became too large with the reconfiguration of the utility's supply. About the best that can be done is to reconfigure flow in the system so that water moves and mixes with water entering the system. Moving and mixing water requires having the distribution system water storage tanks at the right locations in the system in hydraulic terms. As Manual M31, *Distribution System Requirements for Fire Protection* (AWWA, 1998), and most similar references point out, the optimum storage tank location is at the end of the system farthest from the source of supply. Unfortunately, things are more complicated in the real world where little things like topography and wells scattered all over the community sometimes get in the way.

Both Mahwah and Haledon have demonstrated that operating valves or adding components to reconfigure the system can make major improvements.

Haledon is reconfiguring its distribution system to shorten the detention time. Reconfiguring is a very system-specific approach to improving water quality. In the case at hand, by upgrading the pump station at Central Avenue and installing 11,000 linear feet of transmission main, the 575 zone can be split into two zones, the 575 zone and a new 515 zone, each with the storage tank located at the opposite end of the zone from the source of water for the zone. By operating the pumps based upon the water level in each tank, 33% to 50% of the water in each tank can be replaced daily, promoting mixing in the mains as well as the tanks because the piping will experience velocities commensurate with the capacity of the pumps on a daily basis. The probable capital cost of these improvements totals $2.7 million. As an interim improvement, the tank located at the entry point for 515 and 575 zones is sometimes operated on a fill-and-draw basis instead of allowing the tank water level to be equal to the distribution system grade line so that the water can be better mixed.

In Mahwah a large new tank was frequently isolated from the distribution system by its altitude valve because the tank was relatively close to the well fields used most of the time to supply the system. As an interim measure to improve mixing and reduce the age of stored water during the off-peak season, the well fields were shut down every third day and the system supplied from the water in the tank. In 2001, Mahwah added a loop to the system so that closing a valve in the main between the tank and the well fields relocates the tank hydraulically to the opposite end of the system from the well fields.

Both Haledon and Mahwah have instituted tank inspection and cleaning programs to prevent accumulations of sediment from interfering with achieving water quality benchmarks.

SCADA improvements are both infrastructure and management types. The great benefit of current SCADA equipment is its ability to manage more information and support the generation of reports interpreting that information that can then be used to communicate more efficiently with upper management and regulators.

SCADA upgrades are helping Mahwah improve distribution system performance. Mahwah's distribution system has six storage tanks and three booster stations scattered across four pressure zones. The new system routes data from these facilities as well as the water production facilities to a computer located at the central control panel. Operators are able to use the computer to generate reports for regulatory agencies and to address apparent system problems. The long-term plan is to link daily, monthly, and annual reports to minimize data entry and maximize the rollup of information on system characteristics and benchmark performance. Presently, spreadsheet programs are preferred over database programs because spreadsheets require less computer literacy. In the future, Mahwah may progress to more comprehensive asset management and Web-based approaches pioneered elsewhere.

In summary, infrastructure improvements may be evaluated in terms of the following benchmarks:

Benchmark	Goal	Haledon, 2001	Mahwah, 2001
Service outages	26 minutes/year	10	3
Water main breaks	20/100 miles	30	10
Pressure greater than 20 psi	100%	NA	NA
Total coliform bacteria	Zero	Zero	Zero
Total trihalomethanes	< 0.1 mg/L	0.106	0.006
Lead, 90th percentile site	< 0.015 mg/L	0.005	0.020
Unaccounted-for water	15%	20	NA
Cost per customer, $	NA	100	100

MANAGEMENT

Proactive management is the key to improving distribution system performance, as stated above. Three capacities or abilities that distribution system managers especially need today are:

- Improving employee skills
- Communicating information to stakeholders
- Utilizing performance measures

Employees need new skills to adapt to customer expectations and changing technologies. Among these new skills are:

- Computer literacy
- Data interpretation
- Customer relations

Computers are the tools of the future. Superintendents and foremen at a minimum should be comfortable with personal computers, especially spreadsheet applications, because computers are labor-saving devices needed to manage information essential to achieving customer satisfaction.

Data are best managed and interpreted by staff. The author has worked with the staffs of the Mahwah and Haledon utilities to create databases for system water quality monitoring data, system water production data, valve maintenance, and flushing programs. Many additional applications are anticipated, but the limitation on implementation is the education and enthusiasm of employees to organize the information and put it to work to make their time more productive in terms of accomplishing organizational goals.

Employees need to be compassionate to customers perceiving problems with the system. Customer expectations are higher in our increasingly consumer-driven society. Customers may be willing to pay more for service if they are pleased with the service they receive. Likewise, an appreciation for the concerns of top management should translate into providing appropriate information in a timely manner to maintain the good reputation of the utility.

Improving employee skills can be addressed by training existing employees or hiring new employees already possessing the requisite skills. Managers need to concentrate on training existing employees because small and medium-sized utilities are hard-pressed to hire appropriate qualified new employees. New Jersey recently adopted continuing education requirements for licensed water system operators. Haledon and Mahwah employees attended more workshops offering continuing education credits during the past two years than in previous years. Managers need to make education and/or possession of a license a prerequisite for promotion and increased salary.

Distribution system managers are in the information business. Important information includes population served, the number of services, how many miles of pipe, how many valves, how many hydrants, the annual number of main breaks, the annual number of valves and hydrants serviced, etc. The information needs to be communicated to upper management to support the budgetary needs of the system, to engineers and operations specialists responsible for identifying improvement priorities, and to the system's customers. Upper management needs to know the system's objectives and how the budget is being spent. Experience shows that communicating monthly is better than once a year during the budget process. The proper information bases and appropriate employee skills greatly facilitate enhanced communication. Relate accomplishments to costs. Relate both to benchmarks. For example, be able to report the cost per customer compared with similar or nearby utilities. Then, depending upon the account structure and how much work is outsourced, calculate the cost to accomplish various utility functions like scheduled maintenance, unscheduled maintenance, and meter reading. Communicate with customers in municipal newsletters and through the consumer confidence report.

More often than not, the important information is poorly organized if available at all. The failure to be ready to communicate information diverts resources from solving problems to determining what the problems are. Three tools that Malcolm Pirnie uses to assist small or medium-sized utilities handicapped by inadequate communications are operating manuals, CAD system maps, and spreadsheet databases.

Malcolm Pirnie, serving in effect as the utility's engineering department, has compiled critical data into a one-volume binder for quick reference with Haledon and Mahwah. The object is to have information at one's fingertips to allow quick response to operating problems and to upper management. The only investment is for tabs and a binder. The document is anything but pretty, but it satisfies an essential need for small and medium-sized systems.

Malcolm Pirnie has also updated distribution system maps on a CAD platform for Haledon and Mahwah. Most small or medium-sized utilities the author is familiar with outsource system maps because the staff lacks the equipment and training necessary to maintain this essential information. Upgrading to a Geographical Information System (GIS) platform with asset management is very attractive but unrealistic until the utility staff is more capable of maintaining the maps.

Distribution system databases have been created using information generated by SCADA systems, laboratory information systems, and the reference manual for the purposes of capital improvement planning, corrosion control studies, evaluating conservation alternatives, and evaluating strategies to reduce costs. Spreadsheet software facilitates drawing information from diverse sources and analyzing it to generate the information of interest. For example, the service outage benchmark can be rolled up from appropriately formatted reports on individual main breaks. Such data can be manipulated to provide insight to reduce the frequency of main breaks and the cost per break, leading to a rational economic model for the value of infrastructure improvements to reduce main breaks.

CONCLUSIONS

- Management is the key. Decide what to do with the limited time available.
- Information is the tool. Give stakeholders a report card on your stewardship centering on customer service by using key benchmarks.
- Make improvements that advance more than one goal your priority.

ACKNOWLEDGMENTS

The author wishes to thank the organizations of the Borough of Haledon, the Township of Mahwah, and Malcolm Pirnie, Inc., and also the many individuals in these organizations for their support and contributions to the preparation of this chapter.

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CHAPTER · 35

Preventing Water Quality Deterioration in Finished Water Storage Facilities

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BACKGROUND

A recently completed AWWA Research Foundation (AwwaRF)–sponsored research project (Kirmeyer et al., 1999) sought to develop practical guidelines for operating and maintaining finished water storage facilities to prevent water quality problems originating from storage. Project elements included a literature review, a survey of utility practices, a survey of tank inspection firms, as well as case studies and field studies. This chapter summarizes key guidelines and recommendations from the research effort.

The distribution and storage system is viewed as an important barrier in the multiple-barrier approach to protecting the health of water customers. Historically, utilities and regulators have focused on the filtration and disinfection barriers. However, it is increasingly being recognized that the distribution and storage system is as important as other barriers. Over the past 80 years, distribution system deficiencies have accounted for up to 40% of waterborne disease outbreaks in the United States (Kramer et al., 1996; Moore et al., 1994; Herwaldt et al., 1992; Lippy & Waltrip, 1984; Craun & McCabe, 1973).

Water can be thought of as a perishable product with a preservative, packaging, and shelf life. The preservative is the free or combined chlorine residual that is maintained throughout the distribution and storage system. The packaging consists of transmission and distribution system piping and storage reservoirs. The shelf life is the time that water can remain in the distribution and storage system without loss of disinfectant residual or without disinfection by-product formation. Regarding shelf life, storage facilities can have excessive retention time, resulting in a variety of problems. The internal coatings or exposed surfaces within the facility are important packaging that can affect both health and aesthetic parameters. The disinfectant residual, which helps inhibit microbial growth, can begin to dissipate during storage as a result of the long detention times.

Approximately 400,000 finished water storage facilities are currently in service in the United States. According to the AWWA Water Industry Database (AWWA & AwwaRF, 1992), which provides summary information on more than 10,000 finished water storage facilities in the United States, approximately 63% of finished water storage facilities are steel tanks, 15% are concrete tanks, and 19% are belowground reservoirs. About 98% of these storage facilities are covered. The condition of storage facilities has recently come into question in a US Environmental Protection Agency report to Congress (USEPA, 1997). This report estimates a $12 billion tab to maintain and upgrade storage facilities to meet Safe Drinking Water Act regulations over the next 20 years.

STUDY RECOMMENDS FIVE-STEP APPROACH

A utility should consider the following five steps to prevent and/or solve water quality problems in finished water storage facilities.

Step 1. Understand Your Facility

This step is accomplished by developing an assets file, which represents a total historical picture of the storage facility. The assets file may include as-built records, inspection and maintenance records, water quality data, sediment sampling data, operational data, and customer complaint records from the facility's service area. Properly documented and recorded photographs and videotapes can also be very useful in documenting trends in the condition of storage facilities.

The comparison of detailed inspection records, collected at specific intervals, will enable the operator to recognize trends in corrosion rate, sediment accumulation, or coating failure. A single inspection point, although informative, does not allow the operator to establish an optimal period for inspection, cleaning, and painting for a given storage facility. Storage facility condition and operational and maintenance data can be important assets when a utility is troubleshooting problems, planning maintenance intervals, and budgeting for projects.

Step 2. Define the Water Quality Problems

This step is accomplished by monitoring water quality, evaluating sediments and biofilms, and conducting inspections. Comprehensive diagnostic water quality monitoring may need to be conducted to evaluate mixing and dead zones in the facility. Monitoring in the distribution system, evaluating customer complaints, and estimating hydraulic residence time may also be helpful.

Water quality monitoring used to evaluate trends

The US federal drinking water quality regulations do not specifically require the utility to monitor water quality conditions within storage facilities. Likewise, most states do not specifically require monitoring in storage facilities. In comparison, the United Kingdom requires weekly monitoring of storage facilities for bacteria. Table 1 provides the most important drinking water regulations addressing storage facilities. Monitoring beyond the regulatory requirements is advised.

When a monitoring program is developed, a utility should build on a minimum program of monitoring chlorine residual, heterotrophic plate count bacteria, and coliform bacteria. All parameters may not need to be monitored at all facilities, or throughout the year. It is essential that a utility tailor a monitoring program to each storage facility's specific characteristics. Table 2 lists water quality monitoring parameters for consideration and the available sampling options. Storage facilities may be prone to nitrification in systems that practice chloramination, resulting in finished water with elevated nitrite and nitrate levels. Special monitoring is needed to anticipate nitrification and to identify control measures.

Table 1 Water quality parameters and associated regulations for storage facilities

Parameter	Sample Location	Regulatory Limit	Reference	Comments
Disinfectant residual	Entry point to distribution system	0.2 mg/L on a continuous basis	US SWTR*	Only applies to systems using surface water supplies. No UK maximum or minimum limit on disinfectant residual. Residual must be sufficient to ensure appropriate bacteriological quality.
Disinfectant residual or HPC† bacteria	Throughout distribution system	Detectable level of disinfectant residual or HPC bacteria of 500 or less cfu/mL in 95% of samples collected each month for any two consecutive months	US SWTR	Only applies to systems using surface water supplies. In United States, *Legionella* is also regulated by a treatment technique.
	Storage facility	None	UK	Storage facilities are required to be sampled weekly for HPC bacteria.
TTHMs‡	Throughout distribution system	80 μg/L, running annual average based on quarterly samples	D/DBPR,§ Stage 1	Systems serving > 10,000 people that use disinfection
		100 μg/L, rolling three-month average	UK prescribed concentration or value	If fewer than four samples collected annually, no single TTHM sum allowed above

Table continued next page.

Table 1 Water quality parameters and associated regulations for storage facilities *(continued)*

Parameter	Sample Location	Regulatory Limit	Reference	Comments
Haloacetic acids	Throughout distribution system	60 µg/L, running annual average based on quarterly samples	D–DBPR, Stage 1	Systems serving > 10,000 people that use disinfection
Total and fecal coliform bacteria	Throughout distribution system	5% samples positive for total coliform bacteria; repeat samples positive for fecal coliform or *Escherichia coli* bacteria	US Total Coliform Rule	Number of samples determined by population served
	Storage facility		UK	Storage facilities are required to be sampled weekly

[*] SWTR—Surface Water Treatment Rule

[†] HPC—heterotrophic plate count

[‡] TTHMs—total trihalomethanes

[§] D–DBPR—Disinfectants–Disinfection By-products Rule

Table 2 Water quality parameters for finished water storage facilities

Parameter	Purpose	Sampling Procedure Used
Alkalinity	Indicates the potential buffering capacity	On-line ion-selective electrode or grab sample and laboratory analysis
Aluminum	Indicates potential coagulant overfeeding	On-line ion-selective electrode or grab sample and laboratory analysis
Ammonia, total and/or free	Indicates potential for nitrification	On-line ion-selective electrode or grab sample and laboratory analysis
Chlorine residual, total and/or free	Indicates protection from bacterial growth and provides early warning sign of water quality deterioration; monitored at inlet and outlet to control rechlorination when practiced	On-line colorimetric analyzer or grab sample and amperometric titration laboratory analysis
Coliform, total and/or fecal	Indicates presence of indicator bacteria and conformance to maximum contaminant level	Grab sample and laboratory analysis
Conductivity, specific	Can quickly indicate relative changes in total dissolved solids; e.g., alkalinity	On-line ion-selective electrode or grab sample and laboratory analysis
DBPs*	Represents potential for ongoing chemical reactions and DBP formation	Grab sample and laboratory analysis
Heterotrophic bacteria	Provides early warning sign of water quality deterioration	Grab sample and laboratory analysis
Iron	Indicates potential corrosion reactions	On-line ion-selective electrode or grab sample and laboratory analysis

Table continued next page.

Table 2 Water quality parameters for finished water storage facilities *(continued)*

Parameter	Purpose	Sampling Procedure Used
Nitrate	Indicates possibility of nitrification	On-line ion-selective electrode or grab sample and laboratory analysis
Nitrite	Indicates possibility of nitrification	On-line ion-selective electrode or grab sample and laboratory analysis
pH	Indicates changes from the water source; indication of corrosion of concrete or an unlined new facility	On-line ion-selective electrode or grab sample and laboratory analysis
Taste and odor	Evidence of water quality problem in progress	Grab sample and laboratory analysis
Temperature	Differences within storage facility indicate possible stratification and stagnant zones; early warning sign of potential microbial problems	On-line sensor
Turbidity	Provides early warning sign of water quality deterioration	On-line turbidimeter sensor and analyzer

*DBPs—disinfection by-products

A water quality monitoring program cannot be designed to catch or identify every problem. A 1-L (0.26-gal) sample of water in a 38.5-ML (10-mil-gal) storage facility cannot possibly represent the entire contents. Monitoring data provide information on water quality trends.

Sediment monitoring may uncover water quality problems

Storage facility sediments are typically separate from the bulk water column but can potentially affect bulk water quality. Suspended material, such as iron and manganese, turbidity, suspended solids, and precipitated calcium hardness, may settle in storage facilities where the velocities are minimal. Sediment that is resuspended because of flow surges is a potential source of water quality degradation and may contribute to the water's chlorine demand. Though not a regulatory issue, sediment sampling is of use to a utility because it can provide information on causes of water quality problems, such as taste and odor or recurring bacterial counts. Determining the rate of sediment accumulation in a facility can be a useful tool in determining the time between cleanings. Table 3 lists some analytical parameters that utilities may consider for sediment sampling.

The best opportunity for sediment sampling is after drainage and prior to cleaning. One suggested sampling method is a sediment gauge. Operating on the principle of a rain gauge, the sediment gauge is a calibrated cylinder fastened to the bottom of the storage facility. The gauge provides a known sample area, a representative sampling of the surrounding facility floor, calibrated depth measurements, and undisturbed samples following facility drainage. The minimum suggested dimensions would be 12 in. (300 mm) in diameter and 4 in. (100 mm) in height. Using multiple gauges provides comparable samples from different locations in the facility.

Biofilms may also be the source of a water quality problem

Biofilms on tank surfaces can be a potential source of taste and odor problems, loss of disinfectant residual, and recurring bacterial problems. As with sediments, biofilm monitoring is not required by regulation, but it may assist utilities in proactively dealing with potential problems or identifying the causative elements more quickly when problems occur.

Biofilms occur at the water tank interface between the sediment line and the maximum water level. Several areas should be considered for biofilm monitoring—those surfaces always submerged during operation,

Table 3 Example sediment monitoring parameters

Parameter	Indicated by Presence
Iron oxide	Distribution system corrosion
Aluminum hydroxides	Excess aluminum because of after-floc
Calcium carbonates	Supersaturation of minerals in hard waters
Manganese	Source water problem
Heterotophic plate count bacteria	Possible source of taste and odor problems; potential source of recurring bacterial counts
Depth of sediment	Rate of accumulation; possible source of disinfectant residual loss because of resuspension
Gross microbial examination	System cross-connection, poor hydraulic circulation, or failed facility vent screening

the zone in which the water level fluctuates during operation, and the zones of possible stagnation.

Biofilm sampling can be combined with sediment collection following drainage and prior to cleaning. A coupon method is recommended for biofilm sampling. A coupon of standardized size is coated and treated in a fashion identical to the interior tank coating. The coupon is stabilized flush with the tank wall at the sampling location and left in place during operation. During the next cleaning period, the coupon is removed and sent to a laboratory for chemical and microbiological analysis.

Inspections alert the operator to potential water quality problems

Routine, periodic, and comprehensive inspections are important tools in preventing water quality problems. The most common problems identified during inspections are missing or damaged bug screens, nonoperational cathodic protection systems, unsecured hatches, lead paint on interior surfaces, and coatings that have not been approved by NSF International. Contaminant entry through deteriorated bug screens or faulty roof hatches can lead to many problems, including, at worst, a waterborne disease outbreak. For example, a storage tank contaminated with bird droppings containing *Salmonella typhimurium* appears to have caused seven deaths in Missouri (Geldreich, 1996). Unchecked vandalism that creates access to storage could also lead to contamination of the stored water.

Routine inspections are conducted daily or weekly, and periodic inspections are completed every one to four months. The purpose of routine inspections is to monitor the exterior of the storage facility and grounds for evidence of intrusion, vandalism, coating failure, and operational readiness. Periodic inspections are designed to review areas of the storage facility not normally accessible from the ground.

It is recommended that comprehensive inspections be conducted every three to five years for structural condition and possibly more often for water quality purposes. However, the actual inspection frequency should be based on water quality monitoring and the condition of the facility. It appears likely that comprehensive inspections are not being performed at the recommended frequency. With an estimated 400,000 finished water storage facilities in the United States, 80,000 storage facilities should be inspected each year to meet the recommended five-year inspection frequency. On the basis of a project survey of 12 national inspection firms, it is estimated that 6,000 comprehensive inspections are being conducted each year by certified inspectors. It is unlikely that utilities are performing the remaining 74,000 inspections each year.

To properly inspect surfaces, cleaning needs to occur first so that the condition of the surface can be ascertained. Comprehensive inspections should be conducted by a multidisciplinary team of experts representing water quality, maintenance, operations, engineering, and possibly outside specialists. A result of this project's survey of inspection firms was the consensus that a National Association of Corrosion Engineers certified coating inspector was the ideal choice for performing comprehensive inspections.

A comprehensive inspection should, at a minimum, provide a detailed review of the exterior and interior coating condition, concrete foundations and visible footings, structural components, ladders, vents and safety devices, cathodic protection system, overflow pipe, weir boxes, bug screens, and interior sediment depth. Visual and wet inspection methods, including the use of divers and remotely operated vehicles, were reviewed (Lund, 1998; Herwaldt et al., 1992). Float-down inspections are another option. For this option, a boat is placed in the storage facility to enable an inspection to be conducted as the water level is raised or lowered, providing access to higher elevations in the facility.

Step 3. Evaluate Alternatives to Address Water Quality Issues and Select the Best Solutions

This step uses the information from steps 1 and 2 to develop, evaluate, and select the preferred approach to solving or preventing a water quality problem. The alternatives should include maintenance, operations, and engineering design considerations. The solutions or practices should be integrated, and they may include one or all of these elements.

Maintenance alternatives include cleaning, painting, and repairs

Most water quality problems have a maintenance component that contributes to the solution. For example, if a utility experiences a taste and odor problem in a finished water storage facility, maintenance activities may include sediment removal, recoating or additional curing time, or elimination of contamination from external sources. Table 4 summarizes several water quality problems and the possible maintenance tasks that should be performed.

On the basis of the authors' review of state regulations, much discretion is left to the utility for maintenance, disinfection procedures, and approval of coatings; however, many states recommend adhering to AWWA standards, NSF International standard 61, and Ten States Standards (Great Lakes Upper Mississippi River Board, 1997). Most states do not recommend a cleaning frequency, but some states provide guidelines such as "as often as necessary" and "at reasonable intervals."

The results of this project indicate that covered facilities should be cleaned at a minimum of every three to five years, or more often if needed, based on inspections and water quality monitoring. It is also the researchers' judgment that uncovered storage facilities should be cleaned once or twice per year.

Operations alternatives reduce the stored water's age

Excessive water age results from underutilization (i.e., lack of flow) and short-circuiting within the reservoir. Staff members of distribution system operations have two effective tools to reduce water age: turn the water over on a routine basis, and fluctuate the water levels widely. These approaches may run counter to historical methods of reservoir operation and, in many cases, will require a significant change in operating philosophy.

Many utilities have developed their own system operating strategies to improve the quality of water in storage facilities. The Eugene Water and Electric Board (EWEB) in Oregon devised new pumping schemes to improve chlorine residuals and reduce water age in upper-level service reservoirs (Kirmeyer et al., 1999). By synchronizing pump station operations, EWEB found that water could be moved from the first-level service area directly to any of the upper-level service areas without first

Table 4 Possible maintenance actions to solve water quality problems

Water Quality Problem	Items to Check	Possible Maintenance Action
Taste and odors	Check sediment buildup Check condition of coatings If new coating, check for volatile organic chemicals and synthetic organic chemicals Check for external contamination	Remove sediment Repair and/or replace coatings Provide additional curing Replace screens and repair hatches
Loss of disinfectant residual	Check sediment buildup Check condition of coatings Check screens and appurtenances Check for external contamination	Remove sediment Repair and/or replace coatings Repair and/or replace screens and appurtenances Replace screens and repair hatches
Red water or corrosion	Check cathodic protection system Check coatings Check interior steel structure Check sediment buildup	Repair, calibrate, or replace Repair or replace coatings Clean and recoat Clean and remove sediment

being discharged to an intermediate-level service reservoir. The old and new pumping schemes are shown in Figure 1.

Few guidelines on water turnover rates have been published. Experience in Germany suggests a five- to seven-day maximum retention time in reservoirs that have cement-based internal surfaces (Baur & Eisenbart, 1988). Swiss experience suggests a one- to three-day maximum retention time because of lower chlorine residuals carried in that country (Houlmann, 1992).

It should be emphasized that these are theoretical water ages that assume plug-flow conditions. If short-circuiting is severe, the actual water ages within a storage facility can vary significantly from the theoretical and may range from a few minutes to weeks or months. One point is for certain: tanks and reservoirs that ride the system for long periods of time with little or no exchange of water (underutilization) are cause for

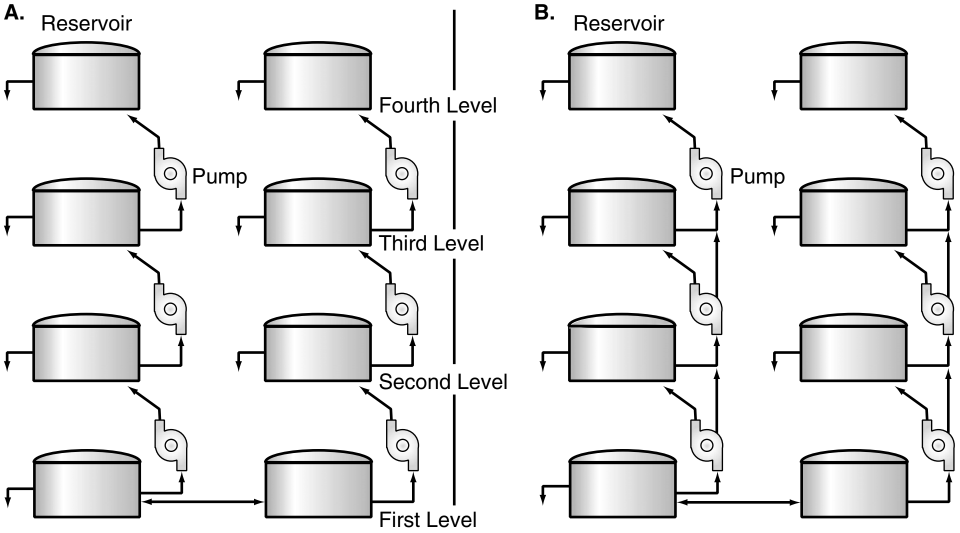

Figure 1 Old (A) and new (B) pumping schemes at the Eugene (Ore.) Water and Electric Board

concern. Both science and field experience indicate that such operation is a recipe for future problems.

To help establish a water age or turnover rate, the utility should consider at the very least two issues: water quality characteristics and reservoir configuration. Each storage facility will have its own needs and characteristics. The utility needs to determine the chlorine decay rate of water in the facility under a variety of water quality conditions. This can be accomplished using bottle tests in the laboratory. With the inlet chlorine residual and the decay rate, the utility can calculate a theoretical turnover rate to maintain a preestablished chlorine residual goal in the outlet for plug-flow or completely mixed reactors. For facilities that are neither plug-flow nor completely mixed reactors, the physical configuration and flow characteristics of the storage facility will dictate mixing in the facility. Modeling and/or tracer studies could be used to predict chlorine levels and water ages. As a starting point and in lieu of site-specific data, the recommended starting point for a theoretical turnover rate is three to five days. Once site-specific evaluations are conducted, the rate can be adjusted.

Some facilities may be configured or operated in such a way that dead zones will be present where water may age for weeks. As a result, fresh water near the inlet may have an adequate chlorine residual, but water in dead zones may have no residual. In such a case, the utility could consider retrofitting the reservoir with a hydraulic circulation system or configuring the inlet to provide momentum mixing of the water. Mixing may lower the overall chlorine residual in the facility, and depending on the extent of the decrease, rechlorination may need to be considered.

In addition to establishing a theoretical turnover rate (i.e., once in three, five, or seven days), the utility may need to establish a water level fluctuation approach that will turn over a majority of the water in one continuous operation. This is especially true for storage facilities with common inlets and outlets, such as standpipes. Simply withdrawing 10% or 20% of the volume of a standpipe each day and immediately refilling could still leave a major portion of storage volume stagnant or poorly mixed for long periods. Thus, if feasible, it would be advisable to fluctuate the water level more widely, with a target withdrawal of 60% of the volume in one day, and then refill it the next. This must be balanced with the need to maintain adequate pressures and emergency storage.

Engineering considerations include sizing, coatings, and mixing

The sizing, number, and type of storage facilities should be selected with water quality in mind. The sizing should be adequate for hydraulic considerations, but being overly conservative or having an extremely long planning horizon often runs counter to maintaining water quality. One possible solution is the phased construction of two smaller reservoirs to facilitate future maintenance.

Paint and coating selection and application procedures should adhere to AWWA standards, and protective coatings should meet NSF International standard 61. The design checklist on pages 721 and 722 provides a list to ensure water quality aspects of storage facilities have been considered.

From a water quality standpoint, perhaps the most important aspect is deciding whether the storage facility will be mixed to preclude dead zones or whether it will operate in a plug-flow mode. This is critical to maintaining a chlorine residual. In most cases for distribution storage facilities, it will be easier to achieve good mixing compared with plug flow. Momentum-based approaches to mixing can be planned by sizing inlets to attain a target velocity, using diffuser pipes, separating inlets and outlets, and other measures. Modeling can be conducted to help predict mixing regimes on existing and new reservoirs at the beginning of design (Clark & Grayman, 1999; Grayman et al., 1999).

One of the more effective mixing approaches can be the use of a hydraulic circulation system, which consists of pumps, diffusers, and

Design Checklist—Finished Water Storage Facilities

	Satisfactorily Addressed?	
Design Guideline	Yes	No
1. Tanks conform to the latest AWWA and NSF International standards as follows:		
a. General design (AWWA D-100 [welded steel], D-103 [bolted tanks, steel and aluminum domed roofs, and coatings], D-110 [prestressed concrete], and D-120 [fiberglass reinforced plastic tanks])		
b. Coatings (AWWA D-102, NSF International 61)		
c. Disinfection (AWWA C-652)		
2. Duplicate storage units are provided when possible to facilitate maintenance.		
3. Tanks are located at least 50 ft (15 m) away from potential sources of contamination.		
4. Tank volume will not result in excessive water age.		
5. Dead storage from vortexing effects or hydraulic effects is minimized.		
6. Capability to isolate tank is provided.		
7. Capability to bypass tank is provided.		
8. The tank drain is not cross-connected to sanitary or storm sewers.		
9. Capability to isolate and bypass altitude valve (if present) is provided.		
10. The tank foundation for above-grade tanks and access roads is at least 3 ft (0.9 m) higher than the 100-year flood elevation.		
11. Foundation drains for below-grade tanks discharge freely at grade.		
12. At least 50% of a below-grade tank is placed above the groundwater table.		
13. Top of tank is at least 2 ft (0.6 m) above finished grade.		
14. Roof is watertight and sloped for drainage.		
15. Vents and overflow pipes are turned downward and equipped with 24-mesh, noncorrodible screens.		
16. Vents are at least 2 ft (0.6 m) above grade or 100-year flood elevation.		
17. Overflow pipe is sized equal to or greater than maximum inflow rate.		
18. Overflow pipe terminates between 1 and 2 ft (0.3 and 0.6 m) above grade and is visible.		

Checklist continued next page.

Design Checklist—Finished Water Storage Facilities

Design Guideline	Satisfactorily Addressed? Yes	No
19. Wall and roof penetrations are welded for steel tanks and equipped with seep rings for concrete tanks.		
20. Inlet and outlet pipes are sized and arranged to promote mixing.		
21. Outlet pipe is equipped with silt stop.		
22. Floating covers and open reservoirs are equipped with bird and small animal deterrents.		
23. Instrumentation (audible and visual) is provided for:		
a. Low water level		
b. High water level		
c. Overflow level		
24. Access hatches have the following features:		
a. Size is 24–15 in. (0.6–0.4 m) minimum.		
b. Frame extends at least 4 in. (100 mm) above tank wall.		
c. Hatch overlaps framed entrance by at least 2 in. (50 mm).		
d. Hatch is hinged.		
e. Hatch is lockable.		
f. Hatch is placed to facilitate periodic cleaning.		
g. Hatch is placed to facilitate batch chlorination.		
25. Safety ladder(s) conform to Occupational Safety & Health Administration standards and prevention of unauthorized use.		
26. Lockable security fence is provided.		
27. Cathodic protection is provided for steel tanks.		
28. Protection from ice damage is provided.		
29. The following monitoring (minimum) parameters are provided:		
a. Measure and record water level.		
b. Measure and record flow rate in and out.		
c. Measure and record chlorine residual.		
30. Multiple dedicated sample taps are provided, or access hatches are designed to allow multiple sampling points within the facility.		
31. Capability to collect samples from inlet and outlet pipes is provided.		
32. Provide security alarms tied to supervisory control and data acquisition system and/or local responding authority as available.		

piping to mix the water within the facility. Energy input through recirculation systems can use velocity gradient G as a starting point. A G value on the order of $10\,s^{-1}$ has been successful in mixing reservoir contents. An additional benefit of the hydraulic circulation system is the means to apply chlorine throughout the storage facility.

Appurtenances on storage facilities, such as vents, hatches, drains, washout piping, sampling taps, overflows, valves, and catwalk, can be critical to maintaining water quality. The Ten State Standards (Great Lakes Upper Mississippi River Board, 1997) provide recommended practices.

Step 4. Implement Good Management Practices and Monitor Effectiveness

This step puts the recommended plan from step 3 into action. This will require coordination and cooperation of various utility staff and possibly outside specialists. Initially, diagnostic monitoring of effectiveness may be required, followed by more routine procedures. Adjustments to improve the results may be needed.

As described in a case study completed for this project, Seattle Public Utilities (SPU) upgraded several storage facilities and improved operational strategies to solve coliform bacteria problems in the distribution system. SPU took advantage of several seismic upgrade projects at storage facilities by implementing design changes to help improve monitoring and to minimize stagnant zones within the storage facilities. As a result of these system improvements, distribution system water quality has improved, as measured by the reduced levels of coliform bacteria.

The implementation of good management practices is also illustrated by a case study completed for this project by Southern California Water Company. The company responded to color, taste, and odor problems in one of its systems by initiating three programs: a reservoir cleaning program, reservoir upgrades, and a change to chloramination. Water quality problems were investigated by reviewing customer complaint records and flushing results and by analyzing water and sediment samples. Implementation of the reservoir cleaning program included coordination among water supply, maintenance, and water quality personnel and attention to regulations addressing confined space, dechlorination, and disinfection practices. The effectiveness of system improvements was monitored by customer complaint records, disinfectant residuals in the distribution system, and sediment sampling in the distribution system.

Step 5. Develop Standard Operating Procedures (SOPs)

According to some water industry professionals, “many water quality problems can be traced to the traditional structure of water utilities, where water quality and treatment have been managed separately from the day-to-day operation and maintenance of the distribution system” (AwwaRF, 1995). The operation of the water distribution and storage system is dependent on several key parties: the utility’s operations department, including system operators and supervisors; the utility management, including its trustees or board of directors; and the water quality department, including laboratory staff. To provide a reliable, efficient operation with an optimum finished water quality, these groups need to understand their role and must communicate effectively with one another.

One way to facilitate better coordination among the various responsible parties is through the use of SOPs. Development of SOPs brings together the multiple responsible parties and opens lines of communication. Written procedures may provide a more efficient response to emergencies and may prevent water quality degradation because of poor practices or miscommunication. Typically, SOPs include a system description with map, facility descriptions, water quality goals, a monitoring plan, a description of the operation’s procedures, a list of responsible parties for each activity, and a list of emergency contact personnel.

STEPWISE APPROACH PROVIDES PRACTICAL SOLUTIONS

This stepwise approach to preventing water quality problems in finished water storage facilities is a practical method for utilities to apply. The importance of developing a storage facility database is emphasized in step 1. Historical data are used to characterize problems, identify solutions, and plan maintenance activities. Step 2 shows how additional information can be generated to further study water quality issues. Water quality monitoring, sediment and biofilm monitoring, and inspections are all important components of this step.

In step 3, multifaceted solutions are developed, including maintenance, operations, and engineering practices. Maintenance solutions include cleaning, painting, and repairs. Members of the operations staff play important roles in minimizing the age of stored water. Operations practices include establishing and implementing turnover rates and varying the water levels. Many storage facilities will need to be upgraded with mixing systems, such as recirculation systems, rechlorination systems, or other means, to maintain water quality. Other engineering considerations are the proper sizing of storage facilities and selections of paints and coatings in accordance with AWWA standards and NSF International standard 61. Step 4 involves the implementation of solutions and requires good management oversight and monitoring. Solutions may evolve as more data are acquired. In step 5, SOPs are developed to facilitate consistent use of best practices by the multiple parties responsible for operating and maintaining the storage facilities.

ACKNOWLEDGMENTS

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CHAPTER · 36

Reduce Customer Complaints Through Control of Water Quality in Distribution System

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BACKGROUND

This chapter shares the "magic" of reducing up to 70% of water quality complaints by improving distribution water quality through various operational controls. Various measures to improve distribution water quality include manganese/iron sequestration by polyphosphates, a controlled unidirectional flushing program, a dead-end flushing program, use of dedicated fire hydrants for water trucks and street sweepers, and VIP customer outreach. After nearly three years, it is evident that significant reductions in the numbers of complaints have been achieved through operational control measures.

INTRODUCTION

The number of customer complaints is a visible measure of quality service for a water purveyor. To reduce the number of complaints is one of the major tasks for a water utilities daily operation, which is crucial to improve customer and community relations. Many water quality issues and problems are associated with the old infrastructure of the water system, and its improvement has been an ongoing project that requires remarkable amounts of capital funding. Is there anything we can do in operations to control the problem and reduce the number of complaints?

Southern California Water Company (SCWC) is an investor-owned public utility, and Central District is one of seven operational districts providing water service in the central and east portions of Los Angeles County. Among the seven systems in Central District, the majority (~90%) of the complaints originate from two systems. Most of these complaints are related to rusty and/or discolored water. By examining the information on well water quality and distribution system pipelines, it has been concluded that the two systems may have different causes of discoloration complaints. In System A, most of the complaints seem related to the levels of manganese (Mn) in the groundwater. For System B, more than 70% of the distribution system is old unlined cast-iron pipes, which may contribute to rusty water complaints.

To resolve numerous customer complaints, "Hot Spot" flushing used to be the only way to respond. However, the outcome of Hot Spot flushing ultimately was not satisfactory. Instead of solving the problem, it was a largely symbolic exercise to calm upset customers. Since providing removal filtration for high-manganese/iron wells requires significant capital investment and is a relatively long process, in 1996 SCWC introduced addition of polyphosphates for sequestration at the wellhead to prevent precipitation of iron/manganese. Although reduction of the number of complaints was noticeable (at least for a certain period of time), a large number of complaints of discolored water continue to be an issue.

IDENTIFYING POSSIBLE CAUSES OF THE PROBLEM

Keep Good Records of Customer Complaints

SCWC has a centralized 24-hour customer service center (CSC) to respond to various customer inquiries and complaints. Customer calls are received at the center by dialing a toll-free number. Trained CSC representatives first attempt to answer customers' questions over the phone. If attention from a field operator, such as a water quality technician, is necessary, a service order is generated and printed out at the local field office. This service order remains in the database linked to the customer's account. Contact with the field operator is also made by telephone and/or pager. The field operator then contacts the customer and handles the inquiry over the phone or makes arrangements to meet the customer on-site if further field investigation is necessary.

After investigating the complaint, the field operator fills out his resolution on the service order along with his name and the date and time. The service order is then returned to a customer service representative to be closed out in the database. The resolved complaint remains linked in the database to the customer account for future reference.

Track Complaint Cause Through Databases

To identify various causes of complaints, several additional databases were created to track the complaints in detail. In addition to the complaints themselves, water quality technicians try to enter the possible causes related to the water source and operational activities. Information gathered includes the age, size, and type of the pipes; source of the serving water; operational changes in the system; and related events such as construction, main breaks, flushing, street sweeping, and supply changes.

Several other databases have also been created to track other related information. The flushing database documents all flushing activities, including flushing velocity, duration, and visual observation of water quality. Other databases include the valve maintenance database, fire hydrant database, and pipeline coupon database.

Although it is impossible to identify the cause for every individual complaint, the data do indicate that quite a significant number of complaints are related to some operation activity such as system flushing, construction use of water, valve maintenance, or fire flow. The combination of rusted cast-iron main, high manganese, and operation activities resulted in rust-colored water in the system.

The table below summarizes complaints with various identified causes for a seven-month period (January to July) in 1998. Also, it provides a basic picture of probable causes of complaints and guidance for corresponding action measures.

Cause	System A	%	System B	%
Flushing (Hot Spot)	56	45%	51	39%
Unknown	40	32%	25	19%
Water truck	12	10%	16	12%
Operation change	7	6%	17	13%
Accident	0	0%	15	12%
Owner internal	9	7%	3	2.5%
Others	0	0%	3	2.5%
TOTAL	124	100%	130	100%

Surprisingly, more than 60% of color complaints were caused by various operations that may be controlled. Flushing alone contributed up to 45% of customer complaints. It is clear that rather than solving the problem, random Hot Spot flushing creates more superfluous complaints.

Therefore, Hot Spot flushing was discontinued in July 1998 and well-planned systematic flushing was piloted for four months from September 1998 to January 1999. A full controlled unidirectional flushing program was implemented in February 1999.

OPERATIONAL CONTROL

After identifying the various causes of complaints, corresponding measures were proposed and implemented. The table below is a brief summary.

Cause	Proposed Action
Flushing (Hot Spot)	• Controlled unidirectional program • Controlled dead-end flushing program
High-manganese well	• Polyphosphates sequestration
Valve O&M	• Notice in the area • Flush first
Operation supply changes	• Flush first
Capital project	• Fill water at designated hydrant • Notice by contractor
Water trucks	• Fill at designated hydrant
Accident	• No control

It is clear that many causes of complaints can be controlled through operational measures. After nearly three years' effort, we have reduced a significant number of complaints by implementing the control measures as discussed in detail in the following sections.

Manganese Sequestration

In System A, three active wells have elevated manganese concentrations. Various color complaints are directly related to the oxidation of dissolved manganese from Mn (2+) to Mn (+4) by chlorine that is dosed for bacteriological disinfection. To prevent oxidation and precipitation of manganese, polyphosphates were introduced in summer 1996 for sequestering manganese. As a result, color complaints were decreased from 350 in 1996 to 190 in 1997 after polyphosphates were introduced (see Figure 1). Since sequestration does not remove manganese from water,

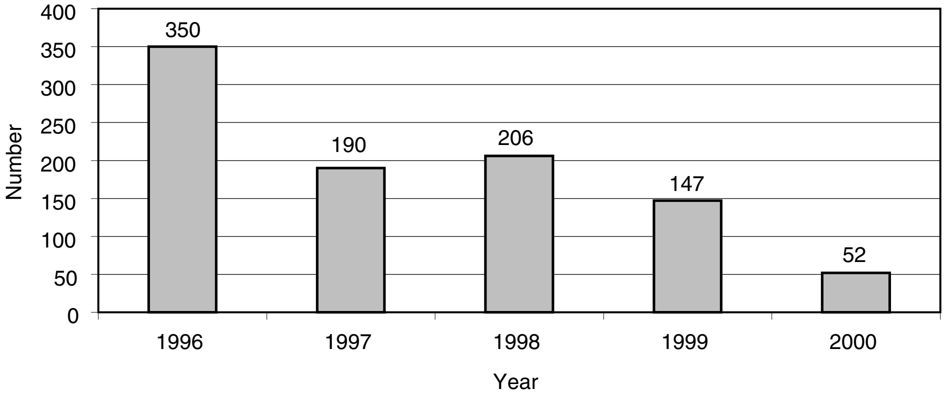

Figure 1 Color complaints in System A

residual manganese will accumulate in the distribution system, especially in dead-end pipelines and the areas in which water has a longer residence time. Therefore, it is important to periodically flush the distribution system to remove the precipitates and implement operational controls to avoid the disturbance of accumulated sediments in old pipelines. As shown in Figure 1, through various operational controls, we were able to further reduce the complaints of discolored water from 206 to 52 (1997–2000).

Since sequestration and oxidation work in opposition to each other for manganese oxidation, it is important to optimize doses of polyphosphates and chlorine to be effective. In general, phosphates should be dosed before chlorine and the chlorine dose should be as low as practical to promote sequestration.

Controlled Flushing Program

As mentioned earlier, the old Hot Spot flushing program created many complaints of dirty water due to the disturbance of sediment in the lines. Uncontrolled flushing (normally with nonconsistent flow velocities) would bring additional particulates from surrounding areas and resulted in spreading the problem instead of relieving it.

A flushing program was piloted in September 1998 and fully implemented in February 1999. The new flushing program is a well-planned, unidirectional systematic flushing with consistent velocity (about 5 feet per second) throughout the system. Isolation gate valves were used to direct water flow, beginning at a source and extending out into the system. A new flushing schedule was established to avoid the disturbance of customer water use. Flushing was scheduled for one evening (Thursday) per week, beginning at 9:00 p.m., to avoid conflicts with customers cooking or doing dishes or laundry.

This flushing strategy required modification of old habits centering on coverage of an area by stressing the proper method of flushing by operating isolation valves, even if it resulted in fewer hydrants flushed in an evening. As a result, the flushing operation successfully cleared large amounts of sediment from the mains while creating very few complaints of dirty water in affected neighborhoods. In addition, the technique of using system valves revealed several instances of valves being broken or closed, restricting flow. These problems were forwarded to the field operations department for corrective action.

Differing from System A, where manganese from wells is one major concern, in System B, old unlined cast-iron pipes were believed to be the major cause of complaints. More than 70% of the distribution pipelines are old unlined cast-iron pipes installed from the 1920s to the 1950s. Most complaints were related to events when the distribution system was disturbed by an accident or operational activities. Figure 2 shows that since the implementation of a controlled flushing program and other operational controls, the number of complaints has decreased significantly.

A separate dead-end flushing program had also been performed. Dead ends are flushed on a regular basis in problem areas, or in response to a complaint. The goal of the dead-end flushing is to purge stagnant water and sediments accumulated with a very low flow velocity (1 to 2 ft/sec). Low-velocity flushing, even during the day, normally does not disrupt water quality in the system as a whole.

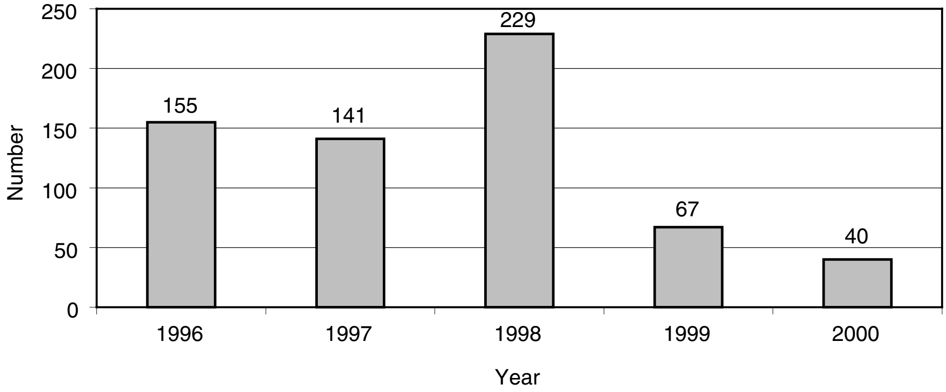

Figure 2 Color complaints in System B

Other Operational Controls

Since it is very hard (if not impossible) to flush out all accumulated precipitates from old cast-iron pipes, any sudden disturbance may cause color complaints. Possible activities include water trucks filling from smaller pipelines such as a contractor for a capital project and street sweeper, operational changes such as changing the source of water, and valve maintenance practice.

Water trucks previously were filled at any convenient fire hydrant. When water was filled at a hydrant that was off a smaller pipeline (2 to 6 in.), flow velocity was much higher than normal flow velocity (1 to 2 ft/sec). Therefore, sediments stirred, resulting in discolored water. To avoid the problem, a group of designated fire hydrants were selected from larger mains (greater than 8 in.) to reduce the flow velocity while filling the truck. Maps with the designated hydrants were prepared and delivered internally and externally. Company customer service representatives consulted the map prior to assigning a hydrant meter to a contractor. The map was also shared with various contractors working for the communities for street cleaning.

To reduce impact, prior flushing was scheduled for any operational events, such as a change in water source, fire flows, and valve and fire hydrant maintenance programs. This control has further reduced the number of complaints. Since all field staff was involved with the flushing program, communication between the departments of water quality, water supply, and field operations have been significantly improved.

VIP Customer Outreach

To monitor distribution water quality, we collect routine distribution samples, including general physical parameters, such as color and odor. However, the samples only represent water quality at the given locations and at a given time. A customer's complaint of water quality is another monitoring datum representing a live situation in the system. Recognizing that customers will not call for all occasions, we proactively started the VIP Outreach Program.

We compiled a list of names of people ("VIPs") who called frequently about water quality, and maintained monthly contact with them by phone to gather information. The majority of our customers have told us they like the fact that we call them on a regular basis. The customers on our VIP list actually help us monitor the water supply and water quality in the distribution system. Some of these people will call us directly during the month to let us know of a possible problem, such as chlorine levels or low water pressure in the distribution system. The VIP list is updated every month and revised periodically. This proactive approach has helped us to maintain accurate information, and also improves public relations with these vocal customers.

SUMMARY

Since September 1998, we have successfully used a series of programs to improve water quality in the distribution system. As a result, water quality complaints have been significantly reduced. For the last three years, a 75% reduction has been achieved for System A, and 83% for System B. Most of these results were achieved with only control of operational activities, not through capital improvement. These programs include:

- Database tracking
- Manganese sequestration
- Controlled systematic flushing program
- Designated fire hydrants for water trucks
- Prior flushing for a fire flow or other operational change
- VIP customer outreach

Team effort is an important factor in this success. The water quality department is working closely with customer service representatives, field operations, and water supply departments to control daily activities to prevent and reduce complaints. The successful experience sets a good example of problem-solving through information gathering, data analyzing, and cooperative team effort. It has also taught us that there are many improvements we can make just through our daily operation.

CHAPTER · 37

Retention Time Management

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BACKGROUND

The water industry has focused predominantly on the quality of treated water and the physical condition of distribution assets when improving the quality of water at the customer's tap. However, the quality of the water delivered is also affected by the time the water is retained in the different elements of the distribution network. This chapter presents work undertaken in the first phase of the AwwaRF research project #2769: *Evaluating Retention Time to Manage Distribution System Water Quality.*

The chapter:

1. Identifies the water quality parameters that are affected by retention time
2. Summarizes the analysis tools available to investigate retention time problems and assesses their utilization via the results of an international survey of 23 utilities
3. Presents methods by which retention time can be adjusted within supply systems
4. Outlines a systematic approach to retention time management

The overall conclusion of the work is that retention time is an effective means of controlling some water quality parameters. However, there is need for a structured approach to help utilities select analytical tools and control measures that are appropriate to the significance of the problem and availability of data and resources. Such an approach was to be developed and tested in Phase 2 of this project.

INTRODUCTION

In 2002, Black & Veatch, in association with Thames Water Utilities, commenced work on AwwaRF research project #2769 with the objective of determining whether water quality within distribution networks can be effectively managed by controlling retention time. This chapter presents the findings from Phase 1 of this two-phase project.

The work focused on addressing the following three questions:

- Is water quality within a distribution system affected by retention time?
- Can retention time within a distribution system be determined?
- Can retention time within a distribution system be altered?

Phase 1 concentrated on scientific theory, technical opinion, and the operational experiences of international utilities. In addressing the three core questions, the current status of knowledge was evaluated on:

- Parameters influenced by retention time
- Analytical tools and methodologies for determining retention time
- The analysis of retention time–related water quality issues
- Engineering and operational solutions for reducing retention time

The extent to which the various analytical tools and control measures are being used by utilities was assessed by circulating a questionnaire to companies who were known to be interested in this approach to manage water quality. Twenty-three utilities from the US, Australia, Denmark, the UK, and the Netherlands returned completed questionnaires. The questionnaire identified the retention time–related problems that were of greatest concern to the water industry, the tools and operational controls currently used to address those problems, and the feasibility of various analytical tools and control measures.

The findings from the questionnaire were presented and discussed at an international workshop attended by delegates from 15 utilities, academics, and the project team. During the workshop, cases in which existing problems had been solved by such methods were presented and the benefits of the options demonstrated. These ranged from simple approaches to calculating retention time to those requiring detailed data analysis and the use of complex software.

WHY IS RETENTION TIME IMPORTANT?

When water leaves a treatment works and passes through a distribution system, its quality, with respect to many chemical and biological parameters, will degrade. The quality of the delivered water will be largely influenced by:

- The quality of treated water supplied into the network
- The condition of distribution assets within the network
- The retention time within the network

The water industry has focused predominantly on the quality of treated water and the physical condition of the distribution assets when improving the quality of water at the customer's tap. However, the quality of the water delivered is also affected by the time the water is retained in the different elements of the system. Both the physical characteristics of the system and its operational regime influence retention time. Physical characteristics such as pipe roughness may change throughout the life of the asset or be modified by rehabilitation. Operational activities may be structured—for example, pump scheduling and planned maintenance—or uncontrolled, as in the case of demand-driven operational responses.

Changes to water quality with time result from the reactions in the bulk water and through the chemical and biological reactions with the distribution system materials with which the water is in contact. The mechanisms by which water quality degrades are very complex and are affected by:

- The chemical and biological makeup of the water
- The mixing of water from different sources
- Hydraulic characteristics
- Dispersion characteristics
- Temperature
- Time
- The material and condition of any surface in contact with the water
- Physical and operational characteristics of the system

The chemical and biological reactions in the bulk water are relatively well understood, thereby enabling models for predicting changes to be developed. The reactions of the water at the pipe-water interface are more complex and less well understood.

Retention time is determined by the design and operation of the network. It is a function of the flow rate and the internal volume of the distribution network. High flow, small-diameter pipes, low storage, and short distances of supply will result in low retention times. Whereas low flows, large-diameter pipes, high storage, and long distances of supply will result in high retention times.

In general, distribution systems are designed to meet the peak diurnal and seasonal demand for water from consumers while providing sufficient additional capacity for firefighting purposes (fire flows). However, in practice the smaller pipe work tends to be sized by minimum fire flow and pressure requirements. In European, and many other international systems, per capita demand and fire flow provision (flow and pressure) are generally significantly lower than in the US (Twort, Ratnayaka, and Brandt, 2000). This allows fire hydrants to be installed on 75–150 mm (3–6 in.) diameter mains, whereas the minimum pipe sizes in the US are commonly 150 mm (6 in.) or 200 mm (8 in.). In the Netherlands, recent revisions to the design criteria provide for the minimum pipe size in new networks to achieve a daily peak velocity of at least 0.4 m/sec (van den Boomen and Vreeburg, 1999), the velocity considered sufficient to prevent sediment accumulation and produce a "self-cleansing" network. This approach has produced new networks comprising 40-mm- (1.5-in.-) and 63-mm- (2.5-in.-) diameter pipes feeding domestic customers in local networks and dead ends where previously 100- or 150-mm- (4- or 6-in.-) diameter pipes would have been installed to deal with fire flows. The network design also incorporates features to ensure that fire flow requirements can be met.

Retention time is relevant to the design of all pipelines. The majority of pipes are designed for fire flows and, when operating under normal demand conditions, will exhibit low velocities even under diurnal or seasonal peak demand conditions, resulting in a long retention time within the system. Conversely, systems designed to achieve self-cleansing velocities will achieve a lower retention time of the water within the system. Irrespective of the pipeline design criteria and the relative magnitude of fire flow provision to peak demands, there are opportunities for reducing retention time by optimizing the operation of new and existing networks, thereby reducing the risk to water quality.

Similarly strategic storage within a system will have a direct impact on the age of water entering the downstream network. The amount of storage provided is generally related to the degree of risk and estimated water requirements to cover the duration of the emergency, such as fire, or failure, such as mains breaks. Long retention time within storage combined with poor circulation within the tank can increase the risk of excessively old water being introduced into a network. It is not only the mean retention time of storage that dictates the age and quality of water passing into the network, but also the mixing characteristics. Some structure designs and pipe arrangements do not take account of water flows and result in the formation of dead zones in which there is poor circulation and pockets of high water age.

The flow in individual pipes and the distance of travel are controlled by the configuration of valves, the routing of flows, and the operational regime for managing sources, storage, and pumping facilities. The practice of subdividing networks into hydraulically discrete areas for leakage management, for example District Meter Areas (DMAs), causes high retention times in the vicinity of the boundary valves to the area. DMA design in the UK often includes an analysis of the age of the water to assess the risk to water quality of the additional closed boundaries and dead ends that are created. The initial design can then be modified to reduce excessive age and reduce the risk of old or stagnant water being supplied to customers.

RETENTION TIME RELATED TO WATER QUALITY PROBLEMS

Although there is considerable information on water quality and the microbiological and chemical parameter changes that take place in distribution systems, the available information can be unclear and site-specific. Difficulties for the water industry arise in finding appropriate practical and pragmatic solutions to distribution quality issues, as many of the factors involved are closely interdependent and action to control one issue may result in a different problem.

The parameters that are affected by retention time include:

- *Microbial water quality:* Despite numerous studies, there is little strong evidence that microbial water quality deteriorates with retention time in the presence of an adequate disinfectant residual. This is because potable water is generally not conducive to the growth of microorganisms. This does not apply to poor-quality waters with a high organic content since the disinfectant residual decays with retention time, leading indirectly to deterioration in microbial water quality (Spiering, 2002).

- *Disinfectant residual and nitrification:* Decay of disinfectant residuals, chlorine and chloramines, and nitrification are strongly related to retention time and organic content of the water. The sensitivity of this relationship is also highly dependent on the material and condition of the distribution pipes (Vasconcelos et al., 1996; Valentine, Ozekin, and Vikesland, 1998; Harrington, 2000).
- *Disinfection by-products (DBPs):* These are influenced by the disinfectant residual, retention time, and other water quality characteristics (e.g., TOC for THM formation or bromine for bromate formation) (Singer, 1999; Khiari et al., 2001).
- *Taste and odor:* Many tastes and odors are due to compounds present in the source water. However, some taste and odor problems may be related to retention time, for example those associated with slow-forming DBPs. Tastes may also become noticeable with time following the decay of chlorine, which may mask less acceptable taste compounds.
- *Discoloration:* One of the causes of discoloration is stagnation and depletion of oxygen in unlined cast-iron mains (Clement et al., 2002). This is particularly the case in the US, where mains are sized to meet much higher fire flow requirements than are common elsewhere. In such circumstances, discoloration is promoted by high retention time.

INVESTIGATING RETENTION TIME WATER QUALITY PROBLEMS

A considerable amount of research has been carried out, much of it under laboratory or pilot-scale conditions, that has increased our understanding of the relationship between retention time and water quality changes. This has allowed the development of two approaches for predicting water quality changes, one based on calculating retention time and the second based on empirical knowledge and water quality data. These two approaches have led to the development of a number of tools for the analysis of water quality in distribution networks.

Tools to evaluate retention time are based on a variety of analytical techniques, including field exercises, laboratory studies, general rules, manual calculations, computer modeling, physical modeling, data visualization, and data analysis. Some tools are already in wide use within the water industry, whereas others are still at a developmental stage. The tools may be divided into four generic types.

1. *Tools for estimating retention time:* For most retention time management techniques, it is necessary to estimate the retention time for a given network configuration. For predictive analyses theoretical retention times can be evaluated for alternative network configurations. The analytical tools available for estimating retention time include:
    - Manual calculations
    - Spreadsheet models
    - Hydraulic modeling techniques (Liou and Kroon, 1987; Boulos et al., 1994; Boulos et al., 1995; Rossman et al., 1993; Reddy, Ormsbee, and Wood, 1995).
    - Computational fluid dynamics (CFD) modeling (Grayman et al., 2000)
    - Tracer studies (Skipworth, Machell, and Saul, 2002)
    - Physical models (Grayman et al., 2000)
2. *Tools for assessing how water quality changes in relation to retention time:* Estimating retention time for current and alternative network configurations will allow operators to see whether it is possible to reduce the retention time significantly. However, it does not determine whether a given retention time is likely to result in acceptable or unacceptable water quality. Hence tools are also needed to assess what the maximum acceptable retention time would be for a particular system. These tools include:
    - Laboratory system simulation techniques used for trihalomethane (THM) formation (APHA, 1994), DBP and biological growth (WRc, 1997), and biological growth and corrosion (Camper et al., 1996; Parent et al., 1996).
    - Pilot distribution systems (laboratory pipe rigs) to study a range of parameters, including biofilms on different materials (Delanoue et al., 1997) and under different hydraulic regimes (McMath et al., 1997; Maier et al., 1999), nitrite formation (Holt et al., 1995), coliforms (McMath et al.,

1999), particle entrainment (Woodward et al., 1995; Maier et al., 1999), and red water generation (Smith et al., 1998).

- Full-scale studies. If the retention time within a network can be readily determined, then the best method of ascertaining the maximum acceptable retention time may be to investigate the response of that network directly.

3. *Tools for predicting water quality changes based on retention time:* It may be appropriate to take the analysis a step further and predict the behavior of different water quality parameters. Most, but not all, of the following techniques involve predicting retention time as part of a more detailed modeling exercise:

    - Batch models, i.e., models that predict water quality changes that occur in a single vessel of water. Some will account for reactions with the walls of the vessel, but most assume the vessel to be inert and thus only model the behavior of the bulk water. Such models have been widely used for bulk water decay of free chlorine (Vasconcelos et al., 1996; Powell et al., 2000) and chloramine (Valentine, Ozekin, and Vikesland, 1998), THMs (Westerhoff et al., 2002; Seidel et al., 2002) and microbial inactivation (Lu, Biswas, and Clark, 1995; Gatel et al., 1995; Servais et al., 1995; Mary-Dile et al., 2000).

    - Storage tank water quality models (Grayman et al., 2000). The internal surface area–to–volume ratio is considerably lower than that for a pipe. Hence wall effects are generally less significant for tank behavior than for networks as a whole. Nevertheless it can still be important for microbial parameters.

    - Network water quality models. This can include using hydraulic models to predict water age and then using the calculated age as an input to a batch model as used by Seidel et al. (2002) to model THM and HAA formation. Alternatively, an integrated hydraulic and water quality model such as EPANET (USEPA, 2002) can be used. Parameters modeled in this way include DBP formation (Grayman et al., 2000; Vasconcelos et al., 1996), microbial (Piriou et al., 1997; Gagnon et al., 1997), and conservative substances blending (Clark, 1991; Geldreich, 1991).

4. *Tools for predicting water quality changes that do not require calculating retention time:* Some tools are based on an understanding of the factors that affect water quality. These do not require calculation of retention time but relate to associations of key parameters that affect water quality; for example, temperature and nutrients. These tools include:
    - On-line sensors
    - System behavior predictors, including statistical modeling techniques, data analysis techniques such as GIS spatial and temporal visualization, and customer surveys
    - Generic rule-based models, which may be rules of thumb, for example the Unified Biofilm Approach developed by van der Kooij et al. (2002), and the "toolbox" approach developed for UKWIR by Chambers et al. (2000) for seven key drinking water parameters
    - Empirical models, including those to simulate chemical dissolution for corrosion control (Rothberg, Tamburini, and Winsor, 2000), blending water (Kippin et al., 2001), and potential of artificial neural networks for analyzing distribution system issues (Rodriguez and Sérodes, 1994)

Utilization of Analysis Tools

Problems can be analyzed using simple manual calculations or computer models and optimization techniques. Provided there are sufficient resources and computing facilities, complex reactions and the interaction between different parameters can be modeled. However, any model requires levels of data and system information commensurate with the degree of accuracy of the required output. Often, adequate data sets are unavailable and this generally leads modelers to make assumptions that simplify the mechanisms to maintain a balance between the availability of data and the required accuracy of the predictive results. Furthermore, while some models have given good results, they are often still too complex to be applied as pragmatic operational tools and may require data that are not readily available to operators.

The conclusions from the survey of international utilities included the recognition of the importance and value of a wide variety of retention time tools for resolving water quality problems. For all but neural networks, over 50% of utilities responded that the analytical tools were considered viable for determining retention time. However, of the 15 tools listed only 7 were already in common use by over 50% of the utilities. The tools that found most frequent application were:

- Review of water quality data
- Hydraulic modeling
- Rules of thumb
- Manual, simple calculations

However, 3 of the 19 respondents assessed that manual calculations were not viable although they had used them; reasons given included "that the method was time intensive" and "that the solution was not very reliable."

It is of interest that the more complex and expensive options involving specialist software, such as reservoir, CFD and water quality statistical modeling, physical models, and GIS, are considered viable but have not been used extensively. Water quality network models are used by just over a third of those who responded to the questionnaire. Only two utilities had used neural network models and optimization software, of which one utility had only used the technique to assess their viability for practical application. All respondents consider water quality data and using hydraulic models as viable and are already using the techniques or are planning to do so. Table 1 summarizes the responses from the 23 international utilities.

METHODS FOR CONTROLLING RETENTION TIME

Many utilities have implemented some forms of retention time management techniques without specifically classifying them as "retention time management." Much can be done to reduce retention time by relatively simple and inexpensive techniques, including flushing at dead ends or reducing the operational top water level of reservoirs. Other techniques involve more significant capital works such as downsizing mains. Where it is appropriate, boundary valve changes to increase velocities or shorten flow paths generally represent the least cost and the most effective solution. Flushing is a short-term palliative that manages a problem, not a long-term solution to an underlying problem. While it is desirable to reduce retention time for water quality reasons, there may be other operational management policies, such as leakage reduction (through pressure reduction), that result in increased retention time. Furthermore, requirements for strategic storage for firefighting and other emergencies conflict with high turnover and low retention times.

Table 1 Feasibility of retention time management analysis tools

	Percentage of the 23 Utilities That Consider the Analytical Tool Viable	
Analytical Tool	Viable	Used
Review water quality data	100	100
Hydraulic network model	100	95
Rules of thumb/practical experience	95	95
Water quality network models	86	38
Manual, simple calculations	81	90
Customer perception surveys	81	67
GIS	81	48
Taste tests	76	71
Tracer studies	71	57
Reservoir model	67	48
Optimization software	62	10
CFD models	62	33
Physical model	52	38
Water quality statistical model	52	14
Neural network models	48	10

Controlling retention time will affect other operational or service-level requirements of the network. These should be considered prior to implementing a proposed change to determine whether the performance characteristics will remain within acceptable limits. In particular, many of the methods will reduce the hydraulic capacity or storage within the network, an issue that needs to be balanced against fire flow and supply pressure requirements.

The methods of controlling retention time may be categorized as either system- or storage-related techniques.

System Methods for Controlling Retention Time

- *Altering the valving within the networks:* Shut valves potentially risk causing low velocities and stagnation in the dead ends they create. Where internal boundaries are necessary, minimize the number of shut valves required to produce the hydraulic boundary and aim to locate valves in streets with high demand on either side of the shut valve. However, in some circumstances shut valves can be beneficial to water quality. Inserting shut valves to lengthen the flow path to a point of high demand will increase the age of water at that demand point, but could significantly reduce the retention time at all the points along the new flow path and that are fed off it.
- *Installing time varying valves:* It may be possible to use time-actuated valves to reduce retention time. For example, hydraulic boundary changes for pressure control may only be required during periods of peak demand. Outside of these times, the boundaries could be left open. Time-actuated valves are also useful for controlling the flow through storage tanks and can be used to ensure a regular cycle of fill and draw.
- *Manual flushing:* Manual flushing is often used as the first remedial measure following a water quality failure (microbial, discolored water, or low chlorine). The primary purpose of flushing is to refresh water quality by expelling the contaminated water. Historically this was most frequently practiced to prevent stagnation in long dead ends that could otherwise result in red water incidents. Following the introduction of the minimum disinfectant residual requirements in the SWTR, regular flushing is practiced by some utilities to maintain the required minimum residual of 0.2 mg/L.
- *Automated flushing:* A limitation of flushing as a means of controlling retention time is that the period between flushings must be shorter than the return period of a water quality violation event. Many water quality parameters may require weekly or even daily flushing, which is expensive in terms of manpower and cost of water. Automated flushing devices have been developed to overcome this limitation.
- *Abandoning mains:* Generally, distribution networks have evolved over many years, invariably resulting in a nonideal network, often with new mains being laid parallel to old mains

to reinforce hydraulic capacity. The old main is frequently retained to maximize hydraulic capacity and to avoid the cost of transferring connections. Since the retention time in each stretch of pipe work of a given material and condition is critical to water quality and not the absolute water age, abandoning a stretch of poor-condition pipe work may alleviate water quality problems even if it does not significantly reduce the overall water age in the network.

- *Downsizing mains:* Most networks have been designed to meet a minimum hydraulic capacity, often with an allowance for future growth or flexibility in the configuration of the network. Pipes tend to be larger than is necessary to meet the daily demand from the network, leading to increased retention times. Hence, there is often scope to replace mains with smaller-diameter pipes but still achieve the required hydraulic capacity.

The Storage Methods for Controlling Retention Time

- *Increasing turnover of storage facilities:* Lack of water turnover in storage facilities has long been recognized as a primary cause of water quality problems within distribution (Kirmeyer et al., 1999). Two aspects that need to be considered are:
  - Mean retention time is directly related to the volume of storage. Hence, it is important to avoid unnecessary storage.
  - Ideally, tanks should be fully mixed. In practice, this is rarely the case and there will be pockets of water that are not well mixed with the bulk of the water, resulting in stagnant zones, and hence the age of water in these zones can be greater than the average age of water in the tank.
- *Adjusting pump schedules:* Storage is often used to balance the variable demand for water from consumers with the output from single-speed pumps that are either on or off. By adjusting the pumping regime, it is often possible to improve the balance between network demand and the supply from the pumps and thereby reduce the volume of storage required. Alternatively, by increasing the duration for which pumps are switched off, it is possible to promote greater turnover in the storage.
- *Reducing the operational top water level of storage facilities:* Where there are no supply pressure implications, reducing the operational top water level is a simple and effective method of

reducing storage, possibly seasonally when demand is reduced, or permanently.

- *Taking storage out of service or removing tank cells:* If storage is not required, then consideration should be given to removing it by removing the entire tank or one or more cells for a multicell facility.
- *Altering reservoir configuration to avoid dead zones:* Storage tanks should be designed and configured to prevent areas of dead water. Pipe inlet and outlet arrangements and baffles can exert a significant influence on flow streams within and turnover of the storage. For multiple cell storage, non-symmetry in the connecting pipework between cells can create selective cell turnover and result in excessive age in the other cells.

Operational Practices for Managing Retention Time

Control measures used for managing retention time were assessed through the survey of international utilities. Each utility was asked to state whether their organization had used or was likely to adopt each technique within the next five years. Table 2 summarizes the range of control measures and the response from utilities as to their use.

Only manual flushing was utilized by greater than 50% of the utilities surveyed. When adding the responses to “Likely to use” the technique, only four of the options were considered realistic by more than 50% of the respondents (manual flushing, adjusting pump schedules, reducing the operational top water level, and altering reservoir storage configuration to remove dead zones). Conversely, of the options, time-variable valves and downsizing pipes were not considered viable by about 50% of the respondents. However, it is worth noting that four of the five Dutch utilities have successfully downsized pipes in their networks.

While nine of the respondents had removed storage capacity (cells or whole tanks) from their systems, nine utilities did not consider it a viable option. However, the overall responses to the three storage techniques indicate that reducing storage and eliminating dead zones within storage are collectively realistic options.

Table 2 Utilization of retention time management techniques

	Likelihood That Companies Will Use the Technique (*number of utilities*)				
Technique	Will Not Use	Unlikely	Possible	Likely	Already in Use
Altering valving in network	1	2	8	3	8
Install time-variable valve	3	8	8	1	2
Manual flushing	1	2	1	2	16
Automatic flushing	3	3	10	6	0
Abandon mains	3	4	6	3	5
Downsize pipes	3	9	2	1	7
Adjust pump schedules	0	2	6	7	7
Reduce the operational TWL	2	4	4	3	9
Remove reservoir cells	4	5	4	0	9
Alter reservoir configuration to remove dead zones	2	2	6	2	10

DISCUSSION AND CONCLUSIONS

A number of important views were also established during the course of the study, in discussion with utilities through the survey, and from the international workshop.

1. There was unanimous agreement that water quality can be improved by managing retention time in distribution networks.
2. The following key water quality parameters need to be investigated further:
   - Chlorination and DBP formation
   - Chloramination and nitrification
   - Discoloration
3. Detailed field measurements should be undertaken before and after interventions in order to validate the effectiveness of improved retention time on water quality.

4. Complex investigations involving hydraulic and water quality modeling have their role, but generally after simple approaches have been exhausted.
5. Small utilities do not have the finances to implement complex tools (such as hydraulic models, CFD). Larger utilities should therefore be targeted to demonstrate the value of these tools. This will enable small utilities to review the potential benefits and justify using retention time tools.
6. Deductive reasoning and common sense can solve many problems. It was therefore recommended that a list of questions be developed that can be used by small operators to identify problems and solutions.
7. There is need for simplified tools (rules of thumb) to manage retention time that operators can follow. This will encourage involvement in retention time issues and give ownership of the approach directly to the people operating the network.
8. There was concern that the wide scope of the problems that could be controlled using retention time management meant that it was not feasible to produce prescriptive guidance.

The survey and workshop demonstrated that many utilities have utilized retention time tools to assist in planning changes to the distribution network and/or to resolve difficult problems. They use the tools to identify where in the network problems occur and to solve a particular problem, and they benefit from the experience gained to provide improved operational practice. Utilities that have used retention time tools for predictive purposes are generally seeking the optimum solution to a current problem, i.e., the tools are being used to assist with planning changes. In most case studies, the utilities have demonstrated benefits, and this has encouraged them to use the same approach to solve similar problems and assist with deriving optimal solutions.

The utility survey, case studies, and workshop discussions reinforce the conclusion that retention time management is potentially capable of controlling a number of water quality parameters. However, there is a need to develop more effective guidance and compare the effectiveness of different techniques. Many tools are available for managing/improving water quality, ranging from simple spreadsheet calculations to complex and expensive software models. The latter are employed mainly by the large utilities for problem solving (reactive) rather than proactively during day-to-day operation of the networks. While the use of retention time tools is increasing in some companies, there has not been widespread acceptance and implementation, particularly in smaller utilities.

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CHAPTER · 38

Pressure Transient Control Strategies to Protect Distribution System Water Quality

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SUMMARY

Pressure transients commonly occur in water distribution systems and can sometimes cause operational problems or system failure. Low pressure transients can cause distribution system contamination if pathways exist for potential contaminants to enter the system. Pathways might include leaks, air valves, and customer service connections. Transient analyses are needed during system design to understand the potential magnitude of pressure transients and, if necessary, to determine how they should be adequately controlled. A transient hydraulic analysis computer program is required to evaluate potential transient conditions in the system and to develop pressure transient control strategies. There are numerous equipment and operational alternatives available for hydraulic engineers to consider when developing a transient control strategy. The best alternative typically depends on the characteristics and transient constraints of the system.

INTRODUCTION

Pressure transients, also commonly referred to as surge or waterhammer, caused by rapid changes in flow rate are common occurrences in pipeline systems. Significant flow rate changes in water distribution systems are typically caused by pump, valve, and hydrant operations; large customer demand changes; and system failures, such as pipeline breaks and power failures. Depending on the magnitude and time period over which the flow rate change occurs, some pressure transients can cause operational problems or system failure. High pressure transients can greatly exceed the normal operating pressure of the system, thereby causing system failure by exceeding the pressure rating of a pipeline, valve, or other system component. High pressure transients that do not exceed the original design pressure ratings of system components can also result in system failure if those components have been weakened over time by corrosion, deterioration of pipe joint materials, improper construction techniques, or other factors. Low pressure transients can cause vacuum conditions that could collapse some pipelines or generate significant "secondary" high pressure transients that occur when air valves are improperly designed or malfunction.

System failures resulting from high pressure transients can cause service interruptions while system operators complete costly emergency repairs. In addition, contaminants from outside of the distribution system can be drawn into the system as a result of vacuum and/or low pressures caused by low pressure transients. This type of system contamination may go undetected by system operators, potentially putting the health of customers at risk.

Some events that cause pressure transients, such as pipeline breaks, are impossible to accurately predict by design engineers and are therefore very difficult to account for during system design. However, other transient initiating events, such as pump and valve operations, should be considered during the design of system facilities. Unfortunately, the importance of transient hydraulic analyses performed by experienced engineers is often underestimated. By neglecting transient hydraulic analyses, adequate high and low pressure transient protection measures may be omitted from the design, potentially resulting in operational problems, system failures, or contamination of the system.

This chapter provides a brief discussion of the potential for system contamination, an overview of the hydraulic transient phenomenon, an approach to analyze hydraulic transients and potential control strategies, and a summary of the transient control strategies implemented in two water systems.

SYSTEM CONTAMINATION POTENTIAL

Two conditions must exist at the same time and location in the distribution system to have the potential for system contamination due to backflow or intrusion. First, there must be a system operational condition that reduces pressure in the system to lower than the pressure outside of the system. Second, there must be a physical condition in the distribution system that provides a pathway for potential contaminants to enter the system.

Normally, pressure in a distribution system is significantly higher than pressure outside of the system. Under these differential pressure conditions, it is impossible for liquids or solids outside of the distribution system to enter the system. However, if a system operational condition results in a sustained or even temporary period of low, zero, or negative pressure in the system, a differential pressure condition will occur that could permit backflow or intrusion into the system. It has been recognized and accepted for a long time that sustained periods of low or zero pressure in distribution systems due to loss of supply, main breaks, system shutdowns, etc., require special precautions, including customer notification and disinfection, before the affected portion of the system is returned to service. These types of sustained low- or zero-pressure conditions are usually recognized by system operators, allowing them to take the appropriate precautions. However, temporary periods of low, zero, or even negative pressure can occur in distribution systems due to low pressure transients, which are not as easily recognized by system operators. Recent research sponsored by AwwaRF involving computer modeling and actual field pressure measurements confirms the occurrence of low and negative transient pressures in distribution systems due to certain system operations. This issue has also been recognized by the USEPA in negotiations on revisions to the Total Coliform Rule.

The second condition that must exist for there to be potential for system contamination due to backflow or intrusion is a pathway. Some common pathways are water main leaks, air valves, and customer service connections. New and properly constructed water mains should not have significant leakage. However, older mains or poorly constructed mains commonly develop leaks with time, providing a pathway for potential system intrusion. Air valves and customer service lines also provide pathways for potential backflow into the system. The potential for system contamination through air valve and service line pathways can be minimized by proper design and maintenance. However, some air valve chambers are commonly flooded with groundwater and/or do not have adequate venting. Also, some customer service lines do not have properly operating check valves or backflow preventers.

Regular leak detection and repair programs, good engineering design, cross connection control and backflow prevention programs, and routine system inspection and maintenance programs can all help to minimize the existence of potential system contamination pathways in a distribution system and therefore should be implemented by water utilities. However, even with these practices in place, some pathways are still likely to exist. If these pathways exist at locations in the distribution system that are susceptible to low and negative pressure transients, there is potential for system contamination due to backflow or intrusion. Therefore, water utilities should also take appropriate precautions to design and operate their system facilities in a manner that reduces the potential for low pressure transients or adequately controls low pressure transients to acceptable levels if they cannot be avoided altogether.

TRANSIENT HYDRAULICS OVERVIEW

This overview is not intended to provide a detailed description and explanation of the complex equations that describe transient hydraulics. Rather, it is intended to provide the reader with a more general description of the hydraulic transient phenomenon. There are numerous publications that provide detailed explanations of transient hydraulic theory, some of which are listed in the "References" section of this chapter.

For "steady" system conditions, there are no changes in flow rate or pressure over time. Steady-state hydraulic analysis is used when flow rates in pipeline systems remain relatively constant or change very gradually. For steady-state analyses of water transmission and distribution systems, water is assumed to be incompressible and pipeline diameters are assumed to remain constant with changing system pressure. While these assumptions are appropriate for steady-state analyses, they are not appropriate for "transient" hydraulic analysis. In reality, water is slightly compressible and pipeline walls expand and contract slightly due to variations in system pressure. Transient hydraulic analyses, which are required when flow rates change rapidly, consider water compressibility and pipeline wall expansions and contractions.

Rapid flow rate changes cause "unbalanced" changes in momentum of the pipeline water column and in forces acting on the pipeline. Because water is slightly compressible and pipeline walls expand and contract, flow rate changes do not occur instantaneously everywhere in the pipeline. For example, if a valve is quickly closed, causing a rapid reduction in flow rate at the upstream end of a pipeline, water at the downstream end of the pipeline does not instantaneously decelerate but continues to flow away from the valve. The rapid change in momentum at the upstream end of the pipeline causes expansion of the water, compression of the pipe wall, and a drop in pressure. This negative "pressure wave" will travel away from the valve at a finite speed, which is dictated by the properties of the fluid (water) and pipeline. Water at the downstream end of the pipeline will only decelerate after the pressure wave reaches the downstream end of the pipeline. If the valve at the upstream end of the pipeline were quickly opened, the resulting pressure wave would be positive.

As mentioned above, the speed at which a pressure wave travels in a pipeline is a function of the fluid properties and the pipeline material, diameter, and wall thickness. Typical wave speeds for water systems with iron pipelines range from 3,000 ft/sec to 4,500 ft/sec depending on the pipeline diameter and wall thickness.

The change in pressure due to a rapid change in flow rate can be estimated using the Joukowsky equation:

$$\Delta H = \Delta V a / g$$

Where:

ΔH = Change in pressure head (ft)
ΔV = Change in velocity (ft/sec)
a = Wave speed (ft/sec)
g = Acceleration of gravity (ft/sec^2)

The Joukowsky equation is useful for estimating the maximum transient pressure change that can result from rapid changes in flow rate. "Rapid" flow rate changes are defined as those that occur in a time period shorter than the system's return period. Return period (T_r) is defined as the length of time it takes for a pressure wave to travel to the end of a pipeline system and back again and can be calculated as follows:

$$T_r = 2L / a$$

Where:

T_r = Return period (sec)
L = Length of pipeline system (ft)
a = Wave speed (ft/sec)

For each return period, the water column flow rate and pressure conditions will change due to pressure wave reflections from the boundaries of the pipeline system. Thus, if the flow rate is changed in a time period greater than T_r, the resulting pressure change will be less than the pressure change calculated by the Joukowsky equation. Therefore, "long" transmission and distribution pipeline systems are much more susceptible to hydraulic transients than "short" systems.

ANALYTICAL APPROACH

The basic equations discussed above can be used to quickly assess the potential for hydraulic transients in a particular water transmission or distribution system. If the potential for significant low and/or high pressure transients exists, a more detailed analysis should be performed using a transient analysis computer program. Such programs are capable of applying the complex differential equations required for transient analysis that are very difficult or nearly impossible to solve by hand calculation. Computer programs allow hydraulic engineers to simulate specific hydraulic transient scenarios and predict the magnitude of potential pressure transients. Transient control alternatives can be evaluated to develop an overall transient control strategy that could include additional or modified system equipment and/or operating procedures. A basic approach to performing a hydraulic transient analysis using a computer program is described below.

Step 1: Develop Computer Model of System

Transient computer models are similar to steady-state computer models. However, additional system information and model detail are needed to simulate transient flow and pressure conditions in the system over time. For example, pipeline segments must be assigned wave speeds. Therefore, available pipeline diameter, material, and thickness class information must be used to estimate wave speed for each pipeline segment in the model. If vacuum conditions occur in the system during the transient analysis, air valve information is needed to accurately model the flow of air into and out of the system. If the operation of a control valve is to be analyzed, hydraulic characteristics for the entire valve stroke, not just the full-open characteristics, are needed. If emergency shutdown of a pump is to be analyzed, additional pump characteristics and motor information are needed to accurately model pump deceleration and potential reverse flow. Information on transient control equipment or facilities, such as surge valves or surge tanks, should also be included in the transient computer model. Similar to steady-state models, transient models should also be calibrated based on flow and pressure data from the actual system when possible.

Step 2: Identify and Analyze Key Transient Scenarios

The operation of system components that can cause significant and rapid changes in flow rate should be reviewed to identify key transient scenarios to be analyzed with the computer model. These operations could include normal shutdown/startup and emergency shutdown of a pumping station and opening, closing, and position adjustments of a control valve. Transient analyses start with an initial steady-state flow and pressure condition in the system. This initial steady-state condition could be a maximum flow condition just prior to a pump shutdown or valve closing, or a minimum flow condition just prior to a pump startup or valve opening. Transient conditions are initiated by the operation of a system component, such as a pump or valve, which is defined and set in the model. The resulting transient flow and pressure conditions simulated by the model can then be evaluated to identify locations within the system where excessive low and/or high pressure transients could occur. A final steady-state flow and pressure condition will eventually occur after the pressure transients have had adequate time to dampen by pipeline friction and transient control equipment. The length of time needed for pressure transients to dampen is also a function of the pipeline system's return period (T_r).

Step 3: Identify and Evaluate Transient Control Alternatives

If the transient model simulation results from step 2 indicate that excessive low and/or high pressure transients can occur in the system, the computer model can then be used to evaluate potential transient control alternatives. Adequate control of pressure transients can sometimes be achieved by modifying the operations or characteristics of a system's flow control components, such as pumps and valves. For example, if the transient analyses indicate that excessive low pressure transients will occur following the closing of a pump control valve, the closing time of the valve can be increased and/or the type of pump control valve can be changed to reduce the magnitude of the resulting low pressure transient. However, in some cases, adequate transient control requires the use of equipment and/or facilities specifically designed to control pressure transients. For example, if the transient analyses indicate that excessive low pressure transients will occur following an emergency shutdown at a pumping station due to loss of power, special transient control equipment or facilities, such as air valves or surge tanks, may be needed to maintain system pressures within the desired limits. There are numerous equipment/facility alternatives for controlling pressure transients. Some of these alternatives are variable-speed pumps, pump control valves, surge relief valves, surge anticipator valves, specialty air valves, and surge tanks. Selection of appropriate alternatives is based on a consideration of effectiveness, reliability, cost, available space, and other factors.

Step 4: Develop Design Criteria for Selected Transient Control Strategy

After an appropriate transient control strategy is selected based on the evaluations performed during step 3, detailed transient control design criteria can be developed based on the transient model results. These design criteria should include key physical specifications and operational procedures for transient control equipment and facilities. Physical specifications should include descriptions of type, size, hydraulic capacity, and pressure/timing settings. Operational procedures should include descriptions of how system equipment and facilities interact. This step is intended to help ensure that design engineers have complete information needed to prepare detailed design drawings and specifications for the selected transient control strategy and that system operators have the information required to operate and maintain the equipment.

CASE STUDIES

The following case studies briefly summarize the analyses performed for two water systems. The case studies are intended to demonstrate different methods utilized to effectively control pressure transients and reduce the potential for system contamination.

Case Study 1

System description

A new 8.5 mgd pumping station is being constructed in the water distribution system to supplement supply to service areas in the northern portion of the distribution system. The station will pump water from service area A, which has a normal hydraulic gradient of 161 ft, to service area B, which has a normal hydraulic gradient of 295 ft. The primary purpose of the pumping station is to transfer water through service area B to another new pumping station that will deliver water to the other northern service areas. Transmission and distribution mains in service area B primarily range in size from 6 in. to 24 in. In addition to the new pumping stations, service area B also includes an elevated tank and a 15 mgd pumping station, which also supplies the service area. Transient analyses were required to assess the impact of potential low and high pressure transients in service areas A and B due to operation of the new pumping station.

Transient constraints

A transient computer model of service areas A and B was developed using system information from an existing steady-state computer model. The transient model was then used to analyze transient events initiated at the new pumping station, including normal pump startup, normal pump shutdown, and emergency pump shutdown (power loss). Model scenarios were established to simulate typical pumping station operating conditions during which significant pressure transients could occur, and to predict the low and high transient pressures that could occur in the system during these events.

In addition to the water system's concerns about generating high pressure transients that exceed the pressure rating of system components, the water system was concerned about the potential for low pressure transients to cause vacuum and/or low pressures in the distribution system directly served by the new pumping station. It is a water system goal to eliminate or minimize the occurrence of low pressure transients that can cause excessively low pressures in the distribution system. By maintaining a minimum distribution system pressure of 20 psi at all times, system customers will always have adequate service pressure. In addition, at this minimum pressure there will always be positive pressure in the distribution system to prevent or minimize the risk of distribution system contamination due to backflow or intrusion.

The constraints of the transient analyses were as follows:

1. Do not exceed the maximum pressure rating of the distribution system pipelines.
2. Maintain a minimum pressure of 20 psi in the distribution system.
3. Minimize rapid and unbalanced forces in the distribution system during transient events.

Transient control strategy

The new pumping station design includes pumps that are equipped with variable frequency drives (VFDs) to allow system operators to set precise pumping station flow rates. The selected transient control strategy uses the speed-ramping capabilities of the VFDs to set appropriate ramping speeds for normal pump startup and shutdown operations, and flow rate adjustment operations that minimize resulting low and high pressure transients.

Additional transient control equipment and/or facilities are needed, however, to control low pressure transients generated by emergency pump shutdown at the new pumping station. One alternative being considered by the water company is the addition of a 5,000-gal pressurized surge tank at the discharge of the new pumping station. This type of surge tank would immediately discharge stored water to the distribution system upon sensing a decrease in system pressure following an emergency pump shutdown. The volume of pressurized air in the surge tank would gradually increase as water exits the tank. The intended transient pressure response is a gradual and limited decrease in system pressure rather than the rapid and excessive decrease in pressure that would occur without a surge tank.

Transient model results

The results of the transient control strategy are shown in Figures 1 and 2. These figures show system pressure versus simulation time for a high-elevation location in the service area B distribution system following an emergency pump shutdown. The minimum system pressure goal of 20 psi is also shown. Due to the pressure transient–dissipating effects of the distribution system, maximum pressure ratings of the system pipelines were not exceeded and are not shown in the figures. As shown in Figure 1, without a surge tank the minimum pressure goal for the system is temporarily not met after the emergency pump shutdown. Addition of a pressurized surge tank maintains system pressures well above 20 psi, as shown in Figure 2. Additionally, the surge tank provides a significant dampening of pressure transients during the transient flow period to help minimize rapid and unbalanced forces in the distribution system pipelines.

Case Study 2

System description

Water supply for this system is treated at a surface water treatment plant (WTP) and delivered to system customers via a water transmission system. The transmission system consists of approximately 26,000 ft of

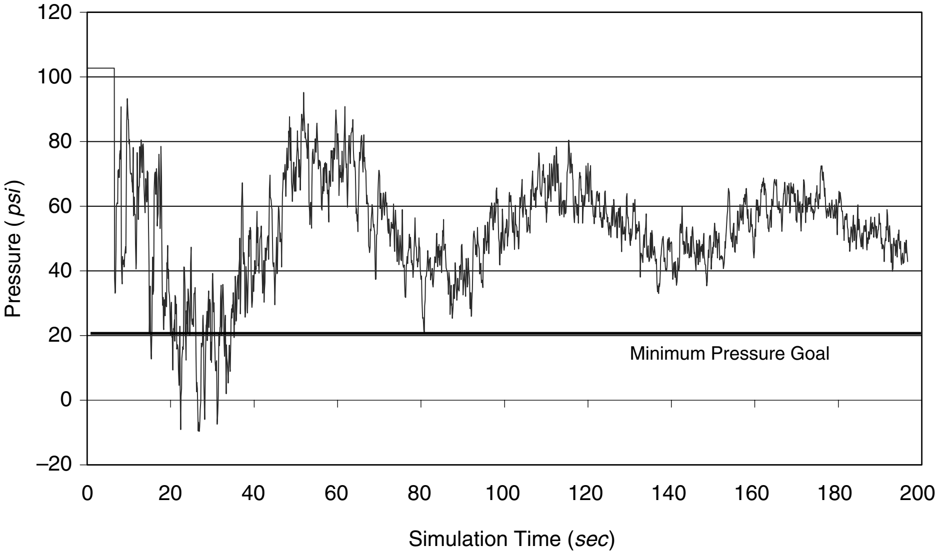

Figure 1 Without surge tank

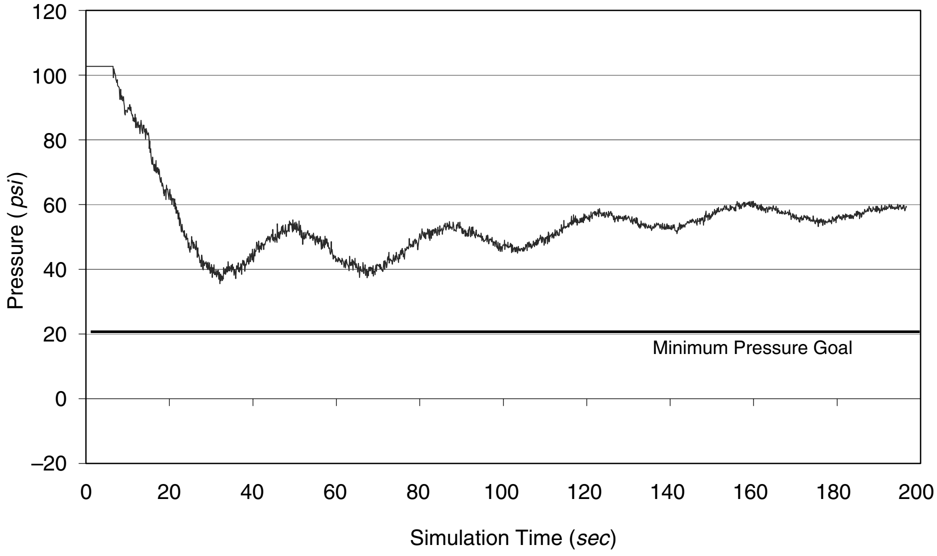

Figure 2 With surge tank

single and parallel 42-in. prestressed concrete cylinder pipe (PCCP) extending from the WTP finished water pumping station to a group of ground storage tanks. Existing flow rates in the transmission system typically range from 35 to 40 mgd. Future flow rates are projected to approach 50 mgd.

Existing transient control equipment includes two 10-in. and four 6-in. hydraulically controlled surge anticipator valves located near the finished water pumping station and air vacuum valves at high-elevation locations along the transmission main. Additionally, an elevated water/air vent pipe at the highest elevation along the transmission main helps to control system pressure during transient events by discharging water from the transmission main during high pressure transients and admitting air into the transmission main during low pressure transients. Most of the existing system transient control equipment is old and/or problematic and was not designed to adequately protect the system at projected maximum flow rates of 50 mgd. Therefore, transient analyses were performed to identify and design system improvements and operating procedures to adequately control potential pressure transients.

Transient constraints

A transient computer model of the transmission system was developed to analyze potential transient events, including emergency pump shutdown (power loss), normal pump shutdown, and normal pump startup for various system flow rates. The computer model was then utilized to develop a transient control strategy to meet the analytical constraints that were identified at the beginning of the transient analyses.

The constraints were as follows:

1. Do not exceed the maximum pressure ratings of the PCCP transmission main.
2. Minimize the volume of water discharged by surge relief valves into the nearby stream to reduce environmental impacts, loss of finished water, and dewatering at the transmission main's high point.
3. Minimize rapid and unbalanced forces in the transmission main during transient events.

Transient control strategy

The selected transient control strategy involves replacing the existing undersized, hydraulically controlled surge anticipator valves with a new surge relief facility consisting of two 8-in. and three 12-in. electronically controlled surge anticipator valves. The new valves are designed to open in response to excessive high and/or low pressures caused by a transient event at the pumping station. The number of valves that open is a function of the system flow rate, as indicated below.

System Flow Rate	Valves Armed
<40 mgd	two 12-in. valves
40–45 mgd	two 12-in. valves and one 8-in. valve
45–50 mgd	two 12-in. valves and two 8-in. valves

The new surge anticipator valves ensure that adequate surge relief capacity is available to control high pressure transients within the maximum pressure rating of the transmission main at varying system flow rates. The "staged" opening also minimizes the volume of water discharged from the surge facility during lower system flow rates when maximum surge relief capacity is not needed to adequately protect the system. Following activation, the valves are designed to close in a staggered manner to minimize secondary high pressure surges that can occur when the surge anticipator valves close. Stroke limiters are furnished with the valves to provide adjustable relief flow rates and to help prevent excessive relief velocities and potential hydraulic "choking."

Other aspects of the transient control strategy include modifications to existing air vacuum valves, replacement of the vent pipe at the transmission main's high point with a specialty air inlet and water relief combination valve, adjustments to opening and closing time settings of the pumping station pump control valves, and development of pump startup and shutdown operating procedures to minimize the high and low pressure transients generated during normal pumping operations.

Transient model results

The results of the transient control strategy are shown in Figures 3 and 4. These figures show system pressure versus simulation time for the critical low point along the transmission main following an emergency pump shutdown when the initial system flow rate is 50 mgd. The system pressure corresponding to the maximum pressure rating of the PCCP transmission main at the critical low point is also indicated. As shown in Figure 3, the maximum pressure rating of the main at the critical point is significantly exceeded following the emergency pump shutdown due to the inadequate hydraulic capacity of the existing surge anticipator valves.

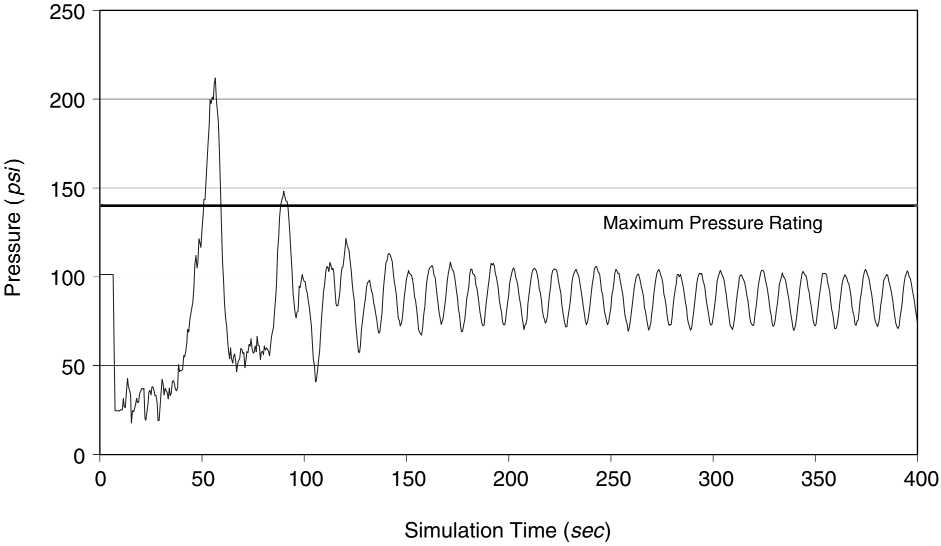

Figure 3 Without surge anticipator valves

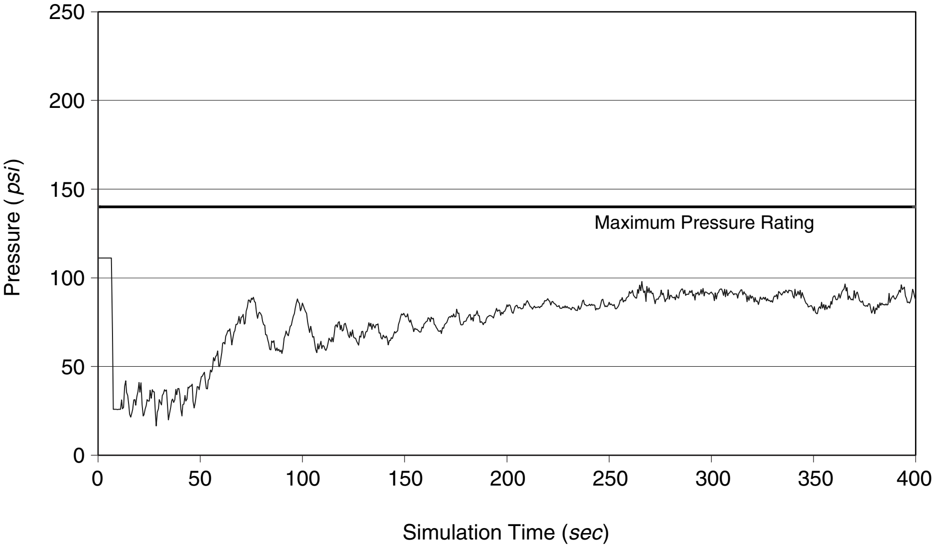

Figure 4 With surge anticipator valves

As shown in Figure 4, the new transient control equipment maintains system pressures well within the maximum pressure rating of the transmission main. Additionally, the new transient control equipment significantly dampens pressure transients to help minimize rapid and unbalanced forces in the transmission main.

CONCLUSIONS AND RECOMMENDATIONS

Pressure transients commonly occur in water distribution systems due to routine and emergency pump, valve, and hydrant operations and other emergency conditions. Low pressure transients can temporarily reduce system pressures in some locations of the system to lower than the pressure that exists outside of the system. If pathways exist at these locations, backflow or intrusion can occur, potentially contaminating the distribution system. Pathways for potential system contamination include leaks, air valves, and customer service connections. Leak detection and repair programs, good engineering design practice, cross-connection control and backflow prevention programs, and proactive system inspection and maintenance programs are effective methods for minimizing potential system contamination pathways. However, it is possible that some pathways will always exist. Therefore, water utilities should design and operate system facilities to control pressure transients resulting from normal and emergency operations to within acceptable levels.

Transient analyses should be performed by hydraulic engineers during system design. Systems that have a significant potential for pressure transients should be analyzed using a transient hydraulic computer program. Transient computer analyses allow the hydraulic engineer to simulate key system operational scenarios to determine if the potential pressure transients can exceed design guidelines. There are numerous methods and alternatives to control pressure transients, including, but not limited to, variable-speed pumps, pump control valves, surge relief valves, surge anticipator valves, specialty air valves, and surge tanks. These alternative methods should be evaluated during the transient computer analyses to develop an appropriate pressure transient control strategy for the system. The best control strategy typically depends on the specific characteristics of the system and the transient constraints established at the beginning of the analysis.

ACKNOWLEDGMENTS

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CHAPTER · 39

Potential for Pathogen Intrusion During Pressure Transients

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BACKGROUND

Pressure transients in drinking water pipelines (i.e., surges) may cause hydraulic pressure gradients, resulting in the potential for intrusion of pathogens present in the external environment into the distribution system. The objectives of this study were to determine the occurrence of indicator microorganisms and pathogens in the vicinity of potable water pipelines and to assess the potential for intrusion attributable to transient distribution system pressure changes. As part of an earlier study (Kirmeyer et al., 2001), soil and water samples were collected at sites immediately exterior to drinking water pipelines at eight locations in six states. Samples were then tested for occurrence of total and fecal coliforms, *Clostridium perfringens, Bacillus subtilis*, coliphage, and enteric viruses. Indicator microorganisms and enteric viruses were detected in more than 50% of the samples examined. Monitoring of pressure transients at a large distribution system indicated that pressure transients occurred frequently, although negative pressures were detected on only one occasion. The results of this study suggest that during negative- or low-pressure events, microorganisms may enter the treated drinking water through pipeline leaks.

From 1971 to 1994, 650 waterborne disease outbreaks and 564,754 cases of gastrointestinal illness were reported in the United States, caused by a microbiological or unknown etiology (Craun & Calderon, 1996). Increasingly, concern has been raised about the adequacy of distribution system barriers. Lippy and Waltrip (1984) found that nearly 37% of community outbreaks from 1946 to 1980 were attributable to failures

A leaky water pipe is located next to a sewer pipe, creating the potential for transient negative pressure to draw water—and pathogens—back into the pipe and thus the distribution system.

of the distribution system. Craun and McCabe (1973) reported that distribution system deficiencies accounted for most of the waterborne hepatitis outbreaks (10 out of 17 outbreaks) in public water systems for the years 1946–1970. Cross-connections and contamination of mains during construction or repair were cited as two of the primary causes of distribution deficiencies. In recent years, waterborne disease outbreaks attributable to distribution system deficiencies have had disastrous consequences. For example, an outbreak of hemorrhagic *Escherichia coli* serotype 0157:H7 occurred in Cabool, Mo., during December 1989 and January 1990 and resulted in 243 cases of diarrhea and 4 deaths (Geldreich, 1996). It was concluded that the illness was caused by waterborne contaminants that entered the distribution system through two major pipe breaks and 43 service meter failures that occurred during unusually cold weather.

DISTRIBUTION SYSTEM VULNERABILITY

Cross-connections

Because of the expansive nature of the distribution system, with its many miles of pipe, storage tanks, and interconnections with industrial users, opportunities for contamination do occur. Cross-connections are recognized as a major risk to water quality, but their probability of occurrence is low because of vigilant cross-connection control programs. Complete assurance of cross-connection control is difficult to achieve, however, and many systems face challenges in maintaining an effective cross-connection control program in the face of dwindling resources and personnel cutbacks.

Main Breaks

Main breaks represent an opportunity for contaminants to enter the distribution system, despite the best efforts to repair these breaks using good sanitary procedures. Utilities typically isolate the affected section, superchlorinate, and flush the repaired pipe. However, flushing velocities may not be able to remove all contaminated debris, and microbiological tests performed to check the final water quality may not detect contaminating organisms. McFeters and co-workers (1986) reported that following the repair of a main break, they found high levels of injured coliform bacteria that were not detectable by standard coliform techniques. Resampling of the site one week later showed persistence of high levels of the coliform bacteria, detectable only using m-T7 agar, a medium specially designed to recover chlorine-injured coliforms.

Backflow Devices

Installation of backflow devices to prevent the entry of contaminated water is an important distribution system barrier. Because of cost considerations, backflow devices are primarily installed on commercial service lines at facilities using potentially hazardous substances (Geldreich, 1996). Examples of such facilities include hospitals, mortuaries, dry cleaners, and industrial users. It is not common for all service connections to have backflow devices, so the possibility of backsiphonage exists at these points. Installation of backflow devices for all service connections would make routine checking of the devices nearly impossible, and without routine inspection, the proper functioning of the units cannot be determined. Furthermore, even when backflow devices have been installed, contamination events have occurred. It was the failure of a backflow check valve that allowed water stored for fire protection to enter the Gideon, Mo., distribution system (Geldreich, 1996). A broken vent in the storage tank allowed birds to enter and contaminate the water with *Salmonella*. Continuous flow of water from the two city water storage tanks, which were also contaminated by pigeons roosting in those tanks, may have contributed to the *Salmonella* outbreak. Three people died because of *Salmonella* infection.

Pressure Transients

Pressure transients in drinking water pipelines are caused by an abrupt change in the velocity of water. These events are also termed "surges" or "water hammer." Pressure transients can be described as waves having both positive and negative amplitude and can draw transient low or negative pressures in a distribution system (LeChevallier, 1999). Because these waves travel through the distribution system, at any point where water is leaking out of the system the transient negative pressure wave can momentarily draw water back into the pipe. Pressure transients can be caused by main breaks, sudden changes in demand, uncontrolled pump starting or stopping, opening and closing of a fire hydrant, power failure, air-valve slam, flushing operations, feed-tank draining, and other conditions.

A general rule of thumb is that for every 1 fps (0.3 m/sec) of velocity forced to a sudden stop, water pressures increase 50–60 psi (345–414 kPa). The opposite is true for a sudden velocity increase, resulting in an instantaneous low or negative pressure (Friedman, 1999). Hydraulic modeling suggests that pressure transients could draw negative pressures, even in a pressurized distribution system (Funk et al., 1999). Because it is not uncommon for water systems to lose more than 10% of total production through leaks in the pipelines (Kirmeyer et al., 2001), opportunities exist for water to intrude through these portals. If pathogens are present exterior to the distribution pipelines, they may enter into the system because of hydraulic pressure gradients or during construction, repair, cross-connections, and conditions or activities in which the system is open to the atmosphere or the environment.

As part of the AWWA Research Foundation study *Pathogen Intrusion Into the Distribution System* (Kirmeyer et al., 2001), soil and water samples were collected immediately exterior to drinking water pipelines at eight locations in six states. Samples were then tested for the occurrence of total and fecal coliforms, *Clostridium perfringens, Bacillus subtilis*, coliphage, and enteric viruses. The objectives of the AwwaRF study were to (1) examine the occurrence of indicator microorganisms and pathogens external to the distribution system and (2) monitor pressure transients at different locations of a large water distribution system using a high-speed pressure data logger.

MATERIALS AND METHODS

Collection of Soil and Water Samples

A total of 65 samples (33 soil samples and 32 water samples) were collected from eight utilities located in six states. The objective was to collect a variety of samples from different locations. Water utilities were instructed to collect soil and water samples whenever distribution system pipelines were exposed for repairs or construction anywhere in the distribution system. These samples were randomly collected from a variety of systems and were intended to represent a sampling of microbial communities exterior to drinking water pipelines.

Sterile trowels and containers were provided to collect soil samples immediately adjacent to the pipeline from an undisturbed portion of the pit. Water samples (if available) were collected from water within the excavation pit. Both samples were shipped in a cooler via overnight carrier to the laboratory for analysis. Both water and soil samples were analyzed for total and fecal coliforms, *C. perfringens, B. subtilis,* bacteriophage, and enteric viruses. Soil samples (10 g) were mixed with 90 mL of sterile reagent-grade water in a sterile container and mixed for 10 min using a magnetic stirrer. Typically, 0.1–10 mL of this solution was transferred quickly (to avoid settling) to microbiological media for analysis.

Bacterial Analyses

Total coliforms were enumerated by the most probable number method (*Standard Methods*, 1995) using lauryl tryptose broth and confirmed using brilliant green lactose broth. Tubes were incubated at 37°C for 24 to 48 hr and scored positive by the production of acid and the formation of gas bubbles in the inverted collection tube. Fecal coliforms were determined by transferring a portion of the positive lauryl tryptose broth to another broth* and incubating for 24 hr at 44.5°C. *B. subtilis* spores were enumerated by heat-treating the sample at 60°C for 15 min in a water bath to kill vegetative cells. After the samples had cooled, they were plated on modified starch plates incubated at 37°C for 24 hr. After 5–10 mL of Lugol's iodine solution was added to the plates, any colony with a clear zone around it was scored as positive for *Bacillus. C. perfringens* spores were enumerated by heat-treating samples in the

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* EC broth, DIFCO, Detroit, Mich.

manner described for *Bacillus* and plating the sample after it had been cooled on mCP agar and incubated at 44.5 ± 0.5°C in an anaerobic jar. After a 24-hr incubation, mCP plates containing yellow, straw-colored colonies were exposed to a concentrated ammonium hydroxide vapor, and colonies that turned pink or magenta were scored positive for *C. perfringens.* Coliphage was determined by the double-layer method described by Adams (1959).

Enterovirus Analyses by Cell Culture Assay

For virus analysis of water samples, 1 L of water was centrifuged at 2,500 × *g* for 10 min, and the supernatant was collected. The water was filtered through a capsule filter.* Viruses were eluted from the filter using 500 mL of 1.5% beef extract (pH 9.4–9.5). The pellet was eluted separately using the method described for soil samples. After elution, the pH of the beef extract was adjusted to 3.5 with 1 *N* hydrochloric acid and stirred for 15 min. The solution was centrifuged for 15 min at 3,290 × *g* using a centrifuge,† and the pellet was retained. The pellet was resuspended in 30 mL of 0.15 *M* sodium phosphate and extracted using 1,1,2-trichlorotrifluoroethane‡ (Abbaszadegan et al., 1999). After low-speed centrifugation (10 min at 1,000 × *g*), the supernatant was aspirated off and penicillin, streptomycin, kanamycin, and amphotericin B were added. The pH was adjusted to 7.0 ± 0.2, and the samples were stored at –80°C until assayed.

For virus analysis of soil samples, 50 g of soil were added to 500 mL of 3% beef extract (pH 9.4–9.5) and stirred for 30 min. The solution was centrifuged at 1,086 × *g* for 30 min, and the supernatant was adjusted to pH 3.5 with 1 *N* hydrochloric acid and stirred for 15 min. The solution was centrifuged for 15 min at 3,290 × *g* and the pellet retained. The pellet was resuspended in 30 mL of 0.15 *M* sodium phosphate and extracted using 1,1,2-trichlorotrifluoroethane. After low-speed centrifugation adjusted to 7.0 ± 0.2, the samples were stored at –80°C until assayed.

Cultural viruses were detected using Buffalo green monkey (BGM) kidney cells grown to confluency in 25-cm^2 (3.9-sq in.) flasks using Eagles minimum essential medium with Earle's salts containing 10% fetal bovine serum. Before exposure to the sample, the growth medium was

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* 1-MDS, Cuno Inc., Meriden, Conn.

† Sorvall, Kendro, Newtown, Conn.

‡ Freon, Dupont, Aldrich Chemical Co., Milwaukee, Wis.

poured off, and the cell monolayer was washed with maintenance medium without serum. For each water, pellet, and soil sample, a 10-mL volume of the final concentrate was added to each of ten 25-cm^2 (3.9-in.^{2}) flasks, incubated at 37°C for 60 min, and rocked every 15 min to facilitate virus adsorption to the cells. Eight milliliters of maintenance medium consisting of Eagles minimum essential medium supplemented with 2% fetal bovine serum was added to each flask, incubated at 37°C, and then examined for viral cytopathic effect (CPE) for 14 days. Any flask with suspected viral CPE was confirmed by inoculation of the medium onto a fresh monolayer of BGM cells and observation for CPE for 14 days. All samples negative for CPE upon the first passage were passed a second time on BGM cells.

Primers Used for Virus Detection

Primers (5′-CCT CCG GCC CCT GAA TG-3′ and 5′-ACC GGA TGG CCA ATC CAA-3′) for the detection of enteroviruses in the sample concentrates produced a 196-base-pair product (Deleon et al., 1990). The hepatitis A virus primers (5′-CAG CAC ATC AGA AAG GTG AG-3′ and 5′-CTC CAG AAT CAT CTC CAA C-3′) produced a 192-base-pair product (Deleon et al., 1990). The Norwalk virus primers (5′-GCAC-CATCTGAGATGGATGT-3′ and 5′-GTTGACACAATCTCATCATC-3′) produced a 206-base-pair product (Moe et al., 1994).

Sample Cleaning for Reverse Transcription–Polymerase Chain Reaction (RT–PCR)

Environmental samples contain humic acid and other substances that may inhibit PCR reaction. Therefore, prior to PCR analysis, each sample concentrate was cleaned before RT–PCR as described by Abbaszadegan et al. (1999) with minor modifications.

The sample concentrate was extracted once with a 5:1 ratio of phenol to chloroform[*] and once with chloroform.[*] The concentrate was combined 1:1 with the phenol–chloroform mixture and vortexed for 3 min. The sample was then centrifuged for 15 min at 14,000 × *g*. The aqueous portion was removed and combined with an equal volume of chloroform, vortexed for 1 min, and centrifuged for 5 min at 14,000 × *g*. The resulting aqueous portion (500 μL) was applied to the top of a column that had been prepared by centrifugation at 400 × *g* for 5 min.

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[*] Amresco Inc., Solon, Ohio

The column consisted of 5 mL of deoxyribonucleic acid– (DNA-) grade purification resin[*] equilibrated in nuclease-free water in a 10-mL syringe plugged at the bottom with a 1-in. (2.5-cm) square piece of sterile tissue.[†] The column was then centrifuged at 400 × *g* for 10 min, and the eluent was collected in a 1.5-mL microcentrifuge tube containing an approximately 50-µL volume of resin.[‡] The microcentrifuge tube containing the resin was vortexed for 1 min and then centrifuged at 1,000 × *g* for 1 min. The supernatant was collected and stored at –80°C until RT–PCR analysis.

RT–PCR

RT–PCR was used to detect ribonucleic acid (RNA) genome enteric viruses. Two RT–PCR reactions were performed on each sample concentrate. One 100-µL reaction (total reaction volume) consisted of 50 µL of the concentrated water or soil sample; in the other 100-µL reaction, 40 µL of sample was seeded with 10 µL of nuclease-free water containing 10 plaque-forming units (pfu) of poliovirus, hepatitis A virus, or 10^{-4} dilution of Norwalk virus cDNA stock. The amount of virus used in the seeded samples was more than what might be expected in an environmental sample; however, the purpose of seeding the concentrates was to provide an indication of whether or not the sample would permit amplification of the viral genome. A sample seeded with fewer viruses may undergo amplification, but the amount of reaction product may be difficult to detect without additional, time-consuming confirmation steps.

Single-tube RT–PCR reactions were performed as follows. The sample was heated at 95°C for 5 min and then placed on ice. A 50-µL reaction cocktail was then added, consisting of 29.4 µL of sterile, nuclease-free water, 10 µL 10 × PCR buffer, 6 µL magnesium chloride (25 m*M*), 1 µL dNTP mix (20 m*M* each dNTP), 1 µL each of forward and reverse primes (30 m*M*), 1 µL RNase inhibitor,[§] 0.1 µL RT[**] (50 units per microliter), and 0.5 µL DNA polymerase[††] (5 units per microliter). The

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[*] Sephadex G-100, Pharmacia Biotech AB, Uppsala, Sweden

[†] Kim-Wipe, Kimberly-Clark Corp., Roswell, Ga.

[‡] Chelex 100, Bio-Rad Laboratories, Hercules, Calif.

[§] RNasin Rnase, PE Applied Biosystem, Foster City, Calif.

[**] SuperScript II, PE Applied Biosystem, Foster City, Calif.

[††] AmpliTaq Gold, PE Applied Biosystem, Foster City, Calif.

tubes were incubated at 42°C for 30 min and then heated to 95°C for 10 min. The reactions were then subjected to 40 temperature cycles of 1 min at 95°C, 1 min at 55°C, and 45 sec at 72°C. A final annealing phase was performed for 7 min at 72°C. The reactions were stored at 4°C until analysis by agarose gel electrophoresis.

Agarose gel electrophoresis was performed using 1.6% agarose I gel[*] containing 0.5 µg/mL of ethidium bromide. The gels were run for 1–2 hr at 100 constant volts and analyzed by photographing the gels exposed to ultraviolet (UV) light using a UV transilluminator.[†]

Physiochemical Analysis

Water and soil samples were sent by overnight carrier to the Belleville (Ill.) Laboratory where they were measured for turbidity and pH. Soil samples were sent to the University of Missouri (Columbia) Soil Testing Laboratory for analysis of moisture content, cation exchange capacity, electrical conductivity, organic matter, and particle size analysis. The cation exchange capacity was estimated from the extractable potassium, calcium, and magnesium and the measure of neutralizable acidity. Electrical conductivity was measured using a modified Wheatstone bridge with alternating currents. Soil organic matter was assessed by measuring the weight loss that resulted from the ignition of organic matter. Particle size analysis was performed using the hydrometer method.

Data Analysis

The distributions of the variables were tested using the univariate analysis.[‡] Spearman correlation was performed for nonnormally distributed data (Cody & Smith, 1991).

Pressure Transients

Pressure transients were monitored at one distribution system that serves about 550,000 people and consisted of 1,600 mi (2,574 km) of distribution main. The system has two water treatment plants that are capable of producing more than 100 mgd (379 ML/d), although the average production is ~75 mgd (284 ML/d). There are 12 storage tanks that can

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[*] Amresco Inc., Solon, Ohio

[†] UVP Inc., Upland, Calif.

[‡] Version 12, SAS Institute Inc., Cary, N.C.

hold a total of 41 mil gal (155 ML). The ground elevation differential is 550 ft (168 m), and peak system pressures typically range between 35 and 270 psi (241 and 1,862 kPa).

Because variations in elevation and system pressure were thought to contribute to the formation of pressure transients, this system was monitored at 12 sites within the distribution system. The sites were selected throughout the distribution system based on the topography, size of main, history of pressure, proximity to pressure-regulating valves, and water uses. Pressure transients were measured at fire hydrants, except for one location that was measured at a hose bib outside a residence and another site that was measured where it connected directly to the pipeline in a valve chamber.

Pressure transients were measured using a pressure data logger.[*] All tests were conducted at a sampling rate of one measurement per second with the exception of site 7, which was sampled at a rate of five measurements per second. Pressure transients were monitored for a duration of 2–43 days.

RESULTS

Indicator and Pathogenic Microorganisms External to the Distribution System

A total of 65 samples (32 water and 33 soil) were assayed from eight water systems in six states to assess the incidence of fecal contamination exterior to drinking water pipelines. Systems were labeled A–I; (system C represents a distribution system that was not included in this part of the study). Samples from each system were numbered accordingly, e.g., system A-1, system B-1. Tables 1 and 2 show the concentrations of microorganisms in water and soil samples; Figure 1 shows the percentage of positive samples.

Total coliforms were detected in 58% (18 of 31) of the water samples and 69.7% (23 of 33) of the soil samples. Total coliform levels ranged from <2 to 1.6×10^3 colony-forming units (cfu)/100 mL in water samples and <2 to 1.6×10^4 cfu/100 g in soil samples. Fecal coliform bacteria were found in 42.8% (12 of 28) of the water samples and 50% (15 of 30) of soil samples. Fecal coliform levels ranged from <2 to 1.6×10^3 cfu/100 mL in water samples and <2 to 1.6×10^4 cfu/100 g in soil samples.

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[*] RDL 1071L/3 Pressure Transient Logger, Radcom Technologies Inc., Mass.

Table 1 Microorganisms in water samples exterior to the distribution system

Sample Identification	Date Sample Received	Total Coliform *MPN*[*]*/100 mL*	Fecal Coliform *MPN/100 mL*	*Clostridium perfringens cfu*[†]*/100 mL*	*Bacillus subtilis cfu/100 mL*	Coliphage *pfu*[‡]*/100 mL*	Culturable Enteroviruses
System B-1	1/27/99	<2	NA[§]	NA	NA	<1	NA
System A-1	2/11/99	<2	NA	NA	1.1×10^5	<1	Negative
System B-2	2/11/99	<2	NA	<1	<1	<1	Negative
System A-2	3/3/99	<2	<2	<1	5×10^3	<1	Positive
System A-3	3/25/99	<2	<2	<1	<1	<1	Negative
System A-4	3/26/99	12	7	5×10^2	3×10^5	<1	Negative
System A-5	4/7/99	2	<2	5×10^2	<1	<1	Negative
System A-6	4/7/99	<2	<2	<1	<1	<1	Negative
System B-3	4/21/99	11	<2	<1	6.2×10^4	<1	Negative
System A-7	5/6/99	5×10^2	30	1×10^3	2.4×10^5	<1	Negative
System A-8	5/11/99	2.8×10^2	<2	<1	9×10^3	<1	Negative
System A-9	5/11/99	4	<2	<1	1.8×10^4	<1	Negative
System A-10	5/13/99	<2	<2	<1	1.5×10^4	1×10^4	Negative
System A-11	5/13/99	<2	<2	<1	5.5×10^4	<1	Negative
System A-12	5/21/99	<2	<2	<1	2×10^4	<1	Negative
System B-4	5/26/99	<2	<2	<1	8.5×10^4	<1	Negative
System B-5	7/28/99	26	<2	1×10^3	6×10^3	2.5×10^2	Positive
System I-1	8/4/99	<2	<2	<1	5×10^2	<1	Negative
System E-1	8/11/99	<2	<2	<1	<1	<1	Negative
System E-2	8/12/99	<2	<2	<1	<1	<1	Negative

Table continued next page.

Table 1 Microorganisms in water samples exterior to the distribution system *(continued)*

Sample Identification	Date Sample Received	Total Coliform *MPN** */100 mL*	Fecal Coliform *MPN/100 mL*	*Clostridium perfringens cfu*† */100 mL*	*Bacillus subtilis cfu/100 mL*	Coliphage *pfu*‡ */100 mL*	Culturable Enteroviruses
System A-13	8/17/99	4	2	<1	3×10^4	<1	Negative
System A-14	8/17/99	5×10^2	70	<1	3.7×10^5	<1	Negative
System D-1	8/18/99	$\geq 1.6 \times 10^3$	3×10^2	<1	3.5×10^4	<1	Negative
System D-2	8/19/99	$\geq 1.6 \times 10^3$	23	<1	3.7×10^4	<1	Negative
System D-3	8/24/99	$\geq 1.6 \times 10^3$	50	1×10^3	4.6×10^6	<1	Negative
System D-4	8/25/99	$\geq 1.6 \times 10^3$	17	2×10^3	1.4×10^5	<1	Negative
System D-5	9/7/99	$\geq 1.6 \times 10^3$	$\geq 1.6 \times 10^3$	5×10^2	TNTC**	<1	Negative
System H-1	9/8/99	NA	NA	NA	NA	NA	NA
System F-1	9/9/99	NS††	NS	NS	NS	NS	NS
System E-3	9/14/99	9×10^2	2	2.5×10^3	1.3×10^6	<1	Negative
System D-6	9/13/99	3×10^2	50	<1	3.7×10^5	<1	Negative
System D-7	9/14/99	50	<2	<1	9.4×10^5	<1	Negative
System G-1	9/15/99	$\geq 1.6 \times 10^3$	$\geq 1.6 \times 10^3$	<1	1.1×10^6	<1	Positive

*MPN—most probable number

†cfu—colony-forming unit

‡pfu—plaque-forming unit

§NA—not assayed

**TNTC—too numerous to count

††NS—no sample

Table 2 Microorganisms in soil samples exterior to the distribution system

Sample Identification	Date Sample Received	Total Coliform *MPN*[*]*/100 mL*	Fecal Coliform *MPN/100 mL*	*Clostridium perfringens* *cfu*[†]*/100 mL*	*Bacillus subtilis* *cfu/100 mL*	Coliphage *pfu*[‡]*/100 mL*	Culturable Enteroviruses
System B-1	1/27/99	<2	NA[§]	NA	NA	<1	NA
System A-1	2/11/99	<2	NA	NA	1.5×10^{6}	<1	Negative
System B-2	2/11/99	<2	NA	<1	4×10^{5}	<1	Negative
System A-2	3/3/99	<2	<2	<1	1.5×10^{7}	<1	Positive
System A-3	3/25/99	1.4×10^{2}	1.4×10^{2}	6.5×10^{4}	1.3×10^{6}	<1	Negative
System A-4	3/26/99	<2	1.7×10^{2}	<1	3×10^{6}	<1	Negative
System A-5	4/7/99	9×10^{2}	20	5×10^{3}	<1	<1	Negative
System A-6	4/7/99	20	<2	5×10^{3}	4.5×10^{5}	<1	Negative
System B-3	4/21/99	1.4×10^{2}	<2	5×10^{3}	9.9×10^{5}	<1	Negative
System A-7	5/6/99	5×10^{2}	<2	<1	1.1×10^{5}	<1	Negative
System A-8	5/11/99	2.4×10^{3}	<2	1×10^{4}	1.9×10^{5}	<1	Negative
System A-9	5/11/99	20	<2	<1	1.6×10^{2}	<1	Negative
System A-10	5/13/99	<2	<2	<1	2.9×10^{5}	<1	Negative
System A-11	5/13/99	<2	<2	1×10^{5}	1.3×10^{6}	<1	Negative
System A-12	5/21/99	1.6×10^{4}	8×10^{2}	<1	1×10^{6}	<1	Positive
System B-4	5/26/99	<2	<2	<1	1×10^{5}	<1	Negative
System B-5	7/28/99	40	<2	<1	6×10^{4}	<1	Negative
System I-1	8/4/99	3×10^{3}	1.4×10^{2}	<1	1.5×10^{7}	<1	Negative
System E-1	8/11/99	<2	<2	<1	1×10^{5}	<1	Negative
System E-2	8/12/99	<2	<2	<1	1.8×10^{5}	<1	Negative

Table continued next page.

Table 2 Microorganisms in soil samples exterior to the distribution system *(continued)*

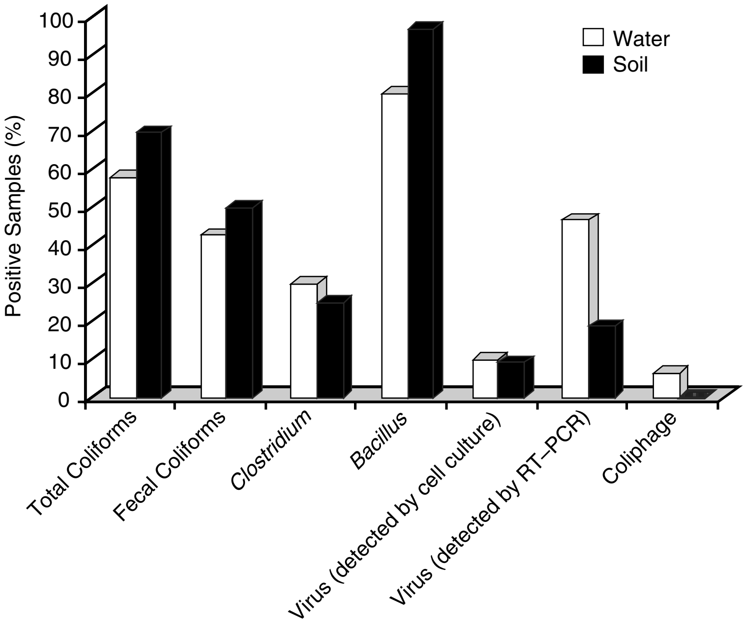
Sample Identification	Date Sample Received	Total Coliform *MPN**/*100 mL*	Fecal Coliform *MPN/100 mL*	*Clostridium perfringens cfu*†/*100 mL*	*Bacillus subtilis cfu/100 mL*	Coliphage *pfu*‡/*100 mL*	Culturable Enteroviruses
System A-13	8/17/99	1.3×10^3	3×10^2	<1	2.5×10^5	<1	Negative
System A-14	8/17/99	7×10^2	1.1×10^3	<1	3×10^5	<1	Negative
System D-1	8/18/99	5×10^3	90	<1	2×10^5	<1	Negative
System D-2	8/19/99	5×10^2	40	<1	6.4×10^6	<1	Negative
System D-3	8/24/99	3×10^3	<2	<1	6.3×10^6	<1	Negative
System D-4	8/25/99	1.7×10^3	<2	2.5×10^4	4.2×10^7	<1	Negative
System D-5	9/7/99	$\geq 1.6 \times 10^4$	1.6×10^4	<1	1.6×10^7	<1	Negative
System H-1	9/8/99	4.8×10^2	40	<1	2.8×10^7	<1	Negative
System F-1	9/9/99	$\geq 1.6 \times 10^4$	1.2×10^2	<1	2×10^7	<1	Negative
System E-3	9/14/99	$\geq 1.6 \times 10^4$	<2	1×10^4	1.3×10^8	<1	Negative
System D-6	9/13/99	1.3×10^3	40	<1	3.8×10^7	<1	Negative
System D-7	9/14/99	5×10^3	20	<1	2×10^6	<1	Negative
System G-1	9/15/99	$\geq 1.6 \times 10^4$	5×10^2	<1	9.5×10^6	<1	Positive

*MPN—most probable number

†cfu—colony-forming unit

‡pfu—plaque-forming unit

§NA—not assayed



Overall, 56.2% of sites were positive for enteric viruses.
RT–PCR—reverse transcription–polymerase chain reaction

Figure 1 Percentage of water and soil samples positive for microbial indicators and viruses

C. perfringens spores were detected in 30% (9 of 30) of water samples and 25% (8 of 32) of soil samples, with levels ranging from below detection to 2.5×10^3 cfu/100 mL and 1×10^5 cfu/100 g, respectively. *Bacillus* spores were frequently encountered (24 of 30 water samples and 31 of 32 soil samples), with soil samples containing as many as 10^8 cfu/100 g *B. subtilis* spores. On the other hand, coliphage was not detected in soil samples and was rarely found (2 of 30) in water samples.

Overall, four sites (12.5% of 32) tested positive for viral infectivity by exhibiting CPE in BGM cells. Three of 32 (9.3%) soil samples and 3 of 30 (10%) water samples were positive for culturable enteric viruses. Two samples (system A-2 and system G-1) were positive for culturable enteric viruses in both soil and water samples. These soil samples also had high concentrations of *Bacillus* spores (10^6–10^7 cfu/100 g). Similarly, culturable enteric viruses were detected in the soil sample of system A-12; that sample also contained high levels of *Bacillus* spores (10^6 cfu/100 g) as well as total and fecal coliform bacteria. The water sample from system B-5 that was positive for culturable viruses had *Bacillus* spore levels of 10^3 cfu/100 mL and was also positive for coliphage.

Sample concentrates were also assayed for viral RNA by RT–PCR using three different pairs of primers specific for enteroviruses, hepatitis A, or Norwalk viruses. Positive control seeded sample reactions were performed to determine whether the sample would inhibit the RT–PCR reaction. If no PCR product was found in the seeded reaction, the sample was deemed inhibitory. Soil samples showed a greater level of RT–PCR inhibition (32 of 96 reactions) than did the water samples (4 of 90 reactions). Overall, 18 of 32 sites (56.2%) were positive by either cell culture or RT–PCR for one of the three viruses tested (Table 3). A total of 14 water samples and 5 soil samples were positive for enterovirus. One water sample was found positive for Norwalk virus, and one soil sample was found positive for hepatitis A virus.

The physiochemical characteristics of soil and water samples are shown in Tables 4 and 5. None of the parameters tested was significantly correlated with the concentration of *Clostridium* or *Bacillus*. However, total coliform concentrations were correlated with percent sand ($p = 0.004$, $n = 33$) and silt ($p = 0.013$, $n = 33$) in soils. Fecal coliform concentrations were correlated with percent sand ($p = 0.044$, $n = 33$) in the soil samples.

Field Validation of Pressure Transients

Table 6 summarizes transient pressure data. Pressure waves were observed at all of the study sites, with variations as high as 90.8 psi (626 kPa). Negative pressures, however, were observed at only one site. The negative pressure was observed following a power outage that shut down a pumping station. The resulting negative pressure lasted 22 sec before static pressures were restored (Figure 2).

DISCUSSION

Pressure Transients

Hydraulic surges are pressure waves that can occur even in pressurized systems. If the amplitude of the wave is large enough, a transient negative pressure can be produced. Circumstances that produce these pressure waves may commonly occur in every water system. Pipeline leaks, although normally under pressure, can be portals by which microorganisms may contaminate finished drinking water. Hydraulic modeling

Table 3 Cell culture and RT–PCR[*] virus results

			PCR						
	Cell Culture		Water			Soil			
Sample Identification	Water	Soil	Enterovirus	Hepatitis A Virus	Norwalk Virus	Enterovirus	Hepatitis A Virus	Norwalk Virus	Water or Soil Positive by Any Method
System B-1	NA[†]	NA	NA	NA	NA	NA	NA	NA	NA
System A-1	–	–	–	–	–	–	–	–	–
System B-2	–	–	–	–	–	IN[‡]	–	–	–
System A-2	+	+	+	–	–	IN	IN	IN	+
System A-3	–	–	–	–	–	IN	IN	–	–
System A-4	–	–	–	IN	–	IN	–	IN	–
System A-5	–	–	–	–	–	IN	IN	–	–
System A-6	–	–	+	–	–	+	–	–	+
System B-3	–	–	+	IN	–	–	–	–	+
System A-7	–	–	–	–	–	–	–	–	–
System A-8	–	–	–	–	–	–	–	–	–
System A-9	–	–	+	–	–	IN	–	–	+
System A-10	–	–	+	–	–	IN	IN	IN	+
System A-11	–	–	–	–	–	IN	IN	IN	–
System A-12	–	+	+	–	–	–	–	–	+

Table continued next page.

Table 3 Cell culture and RT–PCR[*] virus results *(continued)*

			PCR						
	Cell Culture		Water			Soil			
Sample Identification	Water	Soil	Enterovirus	Hepatitis A Virus	Norwalk Virus	Enterovirus	Hepatitis A Virus	Norwalk Virus	Water or Soil Positive by Any Method
System B-4	–	–	+	–	–	–	–	–	+
System B-5	+	–	+	–	+	–	–	–	+
System I-1	–	–	+	–	–	–	–	–	+
System E-1	–	–	–	–	–	+	–	–	+
System E-2	–	–	+	–	–	+	–	–	+
System A-13	–	–	+	–	–	+	–	IN	+
System A-14	–	–	+	–	–	IN	–	–	+
System D-1	–	–	–	–	–	+	–	–	+
System D-2	–	–	–	IN	–	IN	IN	IN	–
System D-3	–	–	–	–	–	–	+	–	+
System D-4	–	–	–	–	–	–	IN	IN	–
System D-5	–	–	–	–	–	IN	IN	IN	–
System H-1	NA	–	NA	NA	NA	–	–	–	–
System F-1	NS[§]	–	NS	NS	NS	IN	IN	–	–
System E-3	–	–	–	–	–	–	IN	–	–

Table continued next page.

Table 3 Cell culture and RT–PCR* virus results *(continued)*

			PCR						
	Cell Culture		Water			Soil			
Sample Identification	Water	Soil	Enterovirus	Hepatitis A Virus	Norwalk Virus	Enterovirus	Hepatitis A Virus	Norwalk Virus	Water or Soil Positive by Any Method
System D-6	–	–	+	–	–	–	–	–	+
System D-7	–	–	+	–	–	IN	–	–	+
System G-1	+	+	IN	–	–	–	–	–	+
Total Positive Samples	3	3	14	0	1	5	1	0	18
§									

*RT–PCR—reverse transcription polymerase chain reaction

†NA—not assayed

‡IN—PCR reaction inhibited

§NS—no sample

Table 4 Physiochemical characteristics of soil samples

Sample Identification	pH	Moisture Content *% Total Weight*	Cation Exchange Capacity *meq/100 g*	Organic Matter *% Total Weight*	Electrical Conductivity *mmhos/cm*
System B-1	NA[*]	21.8	6.1	1.2	0.1
System A-1	7.0	43.25	20.2	2.0	0.5
System B-2	7.0	21.25	5.6	1.7	0.2
System A-2	7.4	61.3	19.2	2.2	1.2
System A-3	7.3	28.3	12.7	1.9	0.4
System A-4	7.5	53.8	21.1	2.5	0.3
System A-5	8.3	39.35	18.0	1.0	0.8
System A-6	7.6	43.85	20.8	1.9	0.5
System B-3	6.7	23.25	3.0	1.8	0.1
System A-7	7.6	40.65	21.0	2.0	0.6
System A-8	7.7	43.15	18.4	1.8	0.4
System A-9	6.9	28.8	14.2	2	2.6
System A-10	7.1	47.05	16.3	3	0.7
System A-11	7.8	33.55	17.3	1.9	0.4
System A-12	6.8	37.7	17.1	1.8	0.9
System B-4	7.2	12.65	2.9	0.7	0.1
System B-5	5.5	23.1	6.4	0.6	0.1
System I-1	7.1	28.7	20.5	1.6	0.4
System E-1	6.8	17.4	9.2	0.7	0.8
System E-2	7.7	19.25	16.4	1.0	0.6
System A-13	8.6	40.6	23.0	1.9	0.7
System A-14	6.2	50.75	16.4	1.5	2.8
System D-1	7.7	35.25	11.1	1.5	0.6
System D-2	6.8	31.8	11.9	1.6	0.3
System D-3	7.0	42.1	9.1	1.2	0.4
System D-4	6.2	32.2	13.1	1.9	0.4
System D-5	7.4	43.95	10.6	1.9	0.4

Table continued next page.

Table 4 Physiochemical characteristics of soil samples *(continued)*

Sample Identification	pH	Moisture Content *% Total Weight*	Cation Exchange Capacity *meq/100 g*	Organic Matter *% Total Weight*	Electrical Conductivity *mmhos/cm*
System H-1	7.0	32.0	10.0	1.0	0.3
System F-1	6.5	20.95	17.3	1.1	0.3
System E-3	7.2	25.5	17.3	1.7	0.3
System D-6	7.3	25.55	18.8	1.6	0.6
System D-7	7.1	37.35	17.7	1.6	0.5
System G-1	7.2	38.4	16.2	2.4	0.6

*NA—not assayed

shows that portions of distribution systems can draw transient negative pressures. Depending on the size of the leaks, the volume of intrusion can range from one to hundreds of gallons (Funk et al., 1999; LeChevallier, 1999).

Pressure transients were monitored at 12 locations in one distribution system using a high-speed pressure data logger to assess the frequency of negative pressure events. Pressure surges were observed at all of the study sites, with variations ranging up to 90 psi (621 kPa). The energy at any point in the pipeline is composed of kinetic and pressure energy. Water will move through a pipe regardless of its position, as long as it is below the energy grade line. Any change in discharge in a pipe (e.g., valve closure, pipe fracture, or pump stoppage) would result in a change of momentum of the flow, creating an impulse force (water hammer). For example, a pipeline may have an open valve located at a distance downstream from a reservoir. If the valve is closed instantaneously, water will ram into the closed gate, creating a pressure shock as it decelerates to zero velocity. The shock wave will travel upstream and downstream from the valve and will ultimately reach the ends of the pipe. If the compressed, halted water in the pipe is not relieved (as in a surge tank), the fluid will start to flow in the opposite direction to relieve the compression. This creates a relief pressure shock that will travel back to the valve. Because the valve is closed, there is no supply for this flow; thus, a negative pressure shock (suction) will be created at the valve

Table 5 Textural classes of soil and physiochemical characteristics of water samples

Sample Identification	Soil			Water		
	% Sand	% Silt	% Clay	Textural Class	pH	Turbidity (*ntu*)
System B-1	72	14	14	Sandy loam	NA*	NA
System A-1	10	34	56	Clay	7.7	5,300
System B-2	76	10	14	Sandy loam	7.7	4,200
System A-2	40	26	34	Clay loam	7.3	180
System A-3	30	34	36	Clay loam	6.7	350
System A-4	10	26	64	Clay	7.9	5,700
System A-5	18	38	44	Silty clay loam	7.6	158
System A-6	12	36	52	Clay	8.1	1,240
System B-3	70	18	12	Sandy loam	6.7	1,440
System A-7	2	30	68	Clay	7.4	3,200
System A-8	12	38	50	Clay	7.6	5,300
System A-9	26	30	44	Clay	7.3	5,300
System A-10	18	30	52	Clay	7.4	1,400
System A-11	34	30	36	Clay loam	7.6	4,600
System A-12	12	36	52	Clay	7.3	40
System B-4	74	16	10	Sandy loam	6.7	6,000
System B-5	84	4	12	Loamy sand	6.4	2,900

Table continued next page.

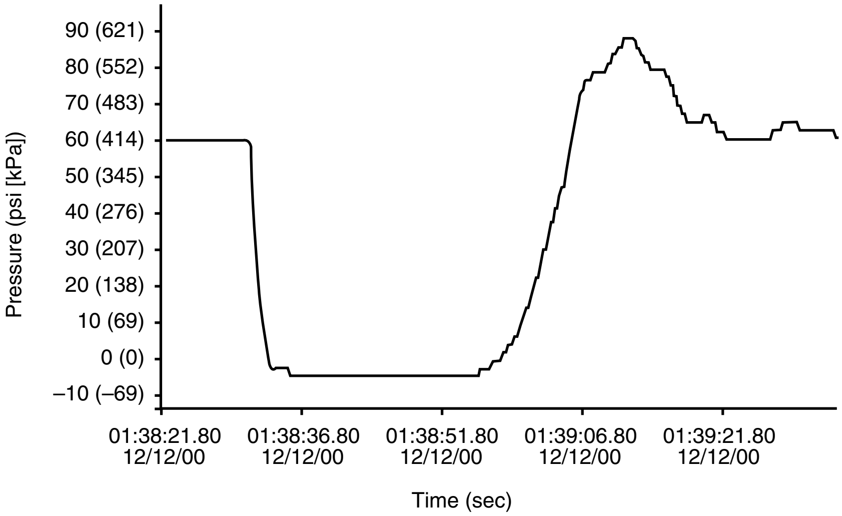
Table 5 Textural classes of soil and physiochemical characteristics of water samples *(continued)*

Sample Identification	Soil			Water		
	% Sand	% Silt	% Clay	Textural Class	pH	Turbidity (*ntu*)
System I-1	46	18	36	Sandy clay	7.4	0.08
System E-1	44	36	20	Loam	6.4	2.6
System E-2	52	30	18	Loam	6.8	63
System A-13	28	26	46	Clay	8.4	5,000
System A-14	16	44	40	Silty clay	6.4	7,300
System D-1	6	60	34	Silty clay loam	8.1	4,700
System D-2	10	58	32	Silty clay loam	6.9	140
System D-3	12	62	26	Silty clay loam	6.1	31,000
System D-4	8	58	34	Silty clay loam	7.1	570
System D-5	10	58	32	Silty clay loam	7.2	15,200
System H-1	56	14	30	Sandy clay loam	7.5	1.7
System F-1	58	18	24	Sandy clay loam	NA	NA
System E-3	40	30	30	Clay loam	7.3	4,700
System D-6	16	54	30	Silty clay loam	7.6	1,640
System D-7	12	56	32	Silty clay loam	6.8	890
System G-1	12	28	60	Clay	7.3	7,100

*NA—not assayed

Table 6 Summary of distribution system water pressures

Site Number	Duration of Monitoring Days	Pressure Monitoring Point	Mean Pressure *psi (kPa)*	Maximum Pressure *psi (kPa)*	Minimum Pressure *psi (kPa)*	Low Pressure Difference From Mean *psi (kPa)*
1	5	Hose bib	21.198 (146.16)	32.420 (223.54)	4.266 (29.41)	16.932 (116.75)
2	5	Fire hydrant	32.173 (221.83)	41.947 (289.23)	20.192 (139.22)	11.981 (82.609)
3	5	Fire hydrant	44.612 (307.60)	51.332 (353.93)	36.200 (249.60)	8.612 (59.38)
4	7	Fire hydrant	98.587 (679.76)	104.086 (717.673)	86.027 (593.16)	12.560 (87.732)
5	2	Fire hydrant	151.672 (1,045.78)	179.449 (1,237.30)	60.859 (419.62)	90.813 (626.16)
6	7	Fire hydrant	174.053 (1,200.10)	180.444 (1,244.16)	163.096 (1,139.23)	10.957 (75.549)
7	5	Fire hydrant	91.936 (633.90)	97.118 (669.63)	81.193 (559.83)	10.797 (74.445)
8	7	Fire hydrant	95.113 (655.80)	97.972 (675.52)	91.857 (633.35)	3.256 (22.45)
9	7	Fire hydrant	174.839 (1,205.51)	238.886 (1,647.12)	167.646 (1,155.92)	7.193 (49.60)
10	7	Fire hydrant	132.233 (911.747)	137.644 (949.055)	111.622 (769.634)	20.611 (142.11)
11	43	Fire hydrant	125.081 (862.433)	154.422 (1,064.74)	90.151 (621.59)	34.930 (240.84)
12	23	Underground vault	61.769 (425.90)	88.018 (606.88)	–4.408 (–30.39)	66.177 (456.29)



The negative pressure lasted approximately 22 sec at 4.4 psi (30.3 kPa) before static pressures were restored.

Figure 2 Record of a negative pressure transient following a power outage

(Simon & Korom, 1997). This shock will travel back and forth until the kinetic energy is dissipated by friction. The process occurs both upstream and downstream from the valve. However, the initial shock will be positive on the upstream side and negative on the downstream side (Simon & Korom, 1997).

In this study, a negative-pressure surge wave was observed following a power outage that suddenly shut down a pumping station. The rapid change in water velocity resulted in a negative pressure that lasted for ~22 sec until static pressures were restored. Ongoing studies are monitoring other water systems to determine the frequency of these negative-pressure events. Negative-pressure events have been observed in other systems when pumps were started or stopped (LeChevallier et al., 2003).

Negative pressures are not necessarily required for intrusion to occur. If the external water table is higher than the depth of the pipeline, atmospheric pressure may be sufficient to overcome low pipeline pressures. For example, 10 ft (3.0 m) of standing water on the distribution pipe would result in an absolute pressure of 19.0 psi (131 kPa) and a gauge pressure of 4.3 psi (30 kPa). Therefore, the low pressure of 4.2 psi (29 kPa) observed at one of the study sites could result in the intrusion of contaminants from outside environments. A survey found that approximately 20% of utilities indicated that 15–30% of the distribution system mains were submerged throughout the year (Kirmeyer et al., 2001). As these data suggest, an intrusion event may occur in low-pressure situations without the presence of negative pressures within the distribution system because conditions exist that may exert an external pressure on pipelines or appurtenances.

Microbial Occurrence

Microbial analysis of a variety of soil and water samples exterior to the pipelines found evidence of fecal pollution and enteric viruses. Fecal coliforms were detected in 42.8% of the water samples and 50% of the soil samples. These findings are not that surprising, given that sewer lines typically are located only a few feet away. Engineering standards call for a horizontal separation of 10 ft (3.0 m) between drinking water and sewer pipelines, but separations can be as little as 18 in. (46 cm) if the pipelines are placed on different levels (*Recommended Standards for Water Works*, 1992). In saturated soil conditions, microorganisms can move several meters in a short period of time (Abu-Ashour et al., 1994). This transport may be aided by water flowing out of the sewer line. The soil and water samples in this study were randomly collected from urban environments, and the location of adjacent sewer lines was not known. A more detailed study could develop better guidelines for the separation of water and sewer mains.

Bacillus spores were detected in 80% and 97% of the water and soil samples, respectively. The high occurrence of *Bacillus* was not unexpected because the organism is a normal flora of soil. The concentration of *Bacillus* spores in soil was as high as 10^8 cfu/100 g of soil, with some of the highest levels associated with samples containing human enteric viruses. It is possible that seepage of sewage stimulated the growth of the soil flora in these locations. The indigenous levels of *Bacillus* spores in natural raw water are between 10^2 and 10^3 cfu/100 mL and are much lower in treated drinking water (Rice et al., 1996). High concentrations of *Bacillus* spores in distribution system water may be considered an indicator of intrusion if their detection was associated with transient-pressure events. Because of the transient nature of the surge event, the volume of intruded water is likely to be extremely small compared with that of the water within the system. It is not likely that monitoring of the drinking water system would capture intrusion events.

The concentrations of total and fecal coliforms were negatively correlated with the percent sand ($p = 0.004$, $n = 33$ and $p = 0.044$, $n = 33$, respectively) in the soil; total coliform concentrations were positively correlated with silt content ($p = 0.013$, $n = 33$) in the soil. These associations may have been related to the increased porosity of sandy soils, resulting in a greater flow of water through soil. The water-holding capacity of sandy soil is low, and it is plausible that a high sand content resulted in increased water percolation. Alternatively, soils with high silt content could retain microorganisms.

Overall, 12.5% (4 out of 32) sampling sites were positive for culturable enteroviruses. The three soil samples that were positive for enterovirus had a high percentage of clay particles and belonged to the textural class of clay-to-clay loam. Viruses can easily become adsorbed to soils and sediments, and the nature of the soil is a major factor influencing virus adsorption (Goyal & Gerba, 1979; Gerba & Schaiberger, 1975). Moreover, clay particles can exert a protective effect against inactivation of viruses (Gerba & Schaiberger, 1975). It is plausible that the clay soils prolonged the survival of viruses in those samples.

Compared with cell culture assay, RT–PCR found a greater level of viral contamination, which is consistent with previous observations (Abbaszadegan et al., 1999). Overall, a total of 18 (56% of 32) samples were virus-positive by either of the methods used (cell culture assay or RT–PCR). All PCR results were confirmed by sequence analysis. A BLAST (Basic Local Alignment Search Tool) search through the National Center for Biotechnology Information in Bethesda, Md., indicated that all the positive enterovirus sequences showed homology with the poliovirus strain Sabin1.

The higher numbers of positive results by RT–PCR could be attributable to several factors: (1) the PCR method has greater sensitivity for the detection of viruses; (2) not all viruses cause CPEs in BGM cells; (3) PCR is able to detect a wider variety of viruses than the cell culture method; and (4) it is possible that PCR detected noninfectious viral nucleic acids. In addition, the minimum detection level of viruses by cell culture method is 1 pfu, which may be equivalent to 1–10 or more virons. On the other hand, RT–PCR can detect as little as a single molecule of RNA. The PCR assay is indicative of the presence of viral nucleic acid, not necessarily the infectious viral particles, and therefore the results should be interpreted as a potential risk of disease rather than an absolute public health concern.

Most of the samples positive for viruses by either cell culture or PCR were also positive for other microbial indicators. Virus-positive samples were also positive for total coliforms (72%), fecal coliforms (50%), *Clostridium* (22%), *Bacillus* (100%), and coliphage (11%), suggesting that indicator organisms may predict the possible presence of pathogens in the vicinity of the water distribution system. Bacterial pathogens were not assayed in this study, so the correlation between the occurrence of indicators and that of bacterial pathogens could not be established.

These results may help explain the data of Payment and colleagues (Payment et al., 1997; Payment et al., 1991), who found that the distribution system was partly responsible for increased levels of gastrointestinal illnesses. Low disinfectant residuals and a vulnerability of the distribution system to pressure transients could account for the viral-like etiology of the illnesses observed. The results of this study emphasize the need to maintain an effective disinfectant residual in all parts of the distribution system and to reduce leakage to minimize the potential for microbial intrusion into potable water supplies.

CONCLUSIONS

Indicator microorganisms and enteric viruses were detected in soil and water samples immediately adjacent to distribution system pipelines. Pressure transients frequently occurred in the distribution system, although negative pressures were detected on only one occasion. The results of this study suggest that opportunities could exist for pathogens to intrude into the distribution system and emphasize the need to maintain an effective disinfectant residual in all parts of the distribution system. Contaminated soil contained up to 10^8 cfu/100 g *B. subtilis* spores, suggesting that this organism may be a useful indicator of intrusion into drinking water supplies. Identification and repair of distribution system leaks are important not only to prevent loss of revenue but also to preserve water quality and safeguard public health.

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CHAPTER · 40

Use of CFD Modeling and Temperature Measurements to Improve Water Mixing Characteristics in Water Storage Tanks

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INTRODUCTION

Historically, water utilities have managed distribution systems primarily to provide reliability of service, such as adequate pressure and fire flow capabilities. However, increasingly stringent regulations such as the Stage 2 D–DBP Rule have added a focus on also maintaining acceptable water quality in the distribution system. Maintenance of distribution system water quality is now a major challenge for many utilities. Water quality challenges in the distribution system may include loss of disinfectant residual, increase in disinfection by-products, biological growth, and for systems that use chloramines, nitrification.

Storage tanks that are underutilized and have poor mixing characteristics represent a significant potential for water quality degradation within the distribution system. Computational fluid dynamic (CFD) modeling and water temperature measurements can be very effective tools to predict mixing characteristics inside the tanks. This chapter focuses on the use of CFD modeling and temperature measurements to determine mixing characteristics in storage tanks, using a case study, the City of Virginia Beach. The results from mixing predictions are verified by water quality data and used to identify the storage tanks with inadequate mixing characteristics and thus the greatest potential for water quality degradation.

Once the nature of the mixing problem is identified, it becomes easier to make the appropriate physical and operational modifications to improve mixing in the storage tanks. This chapter also presents general recommendations for improving mixing, and reports on actual physical modifications that have improved mixing characteristics in a storage tank.

EVALUATION OF MIXING CHARACTERISTICS WITH CFD MODELING

Computational fluid dynamic modeling was performed on two types of tanks to assess the effect of inlet momentum, inlet pipe orientation, and tank geometry on the mixing characteristics of the water inside the tanks. The tank types are:

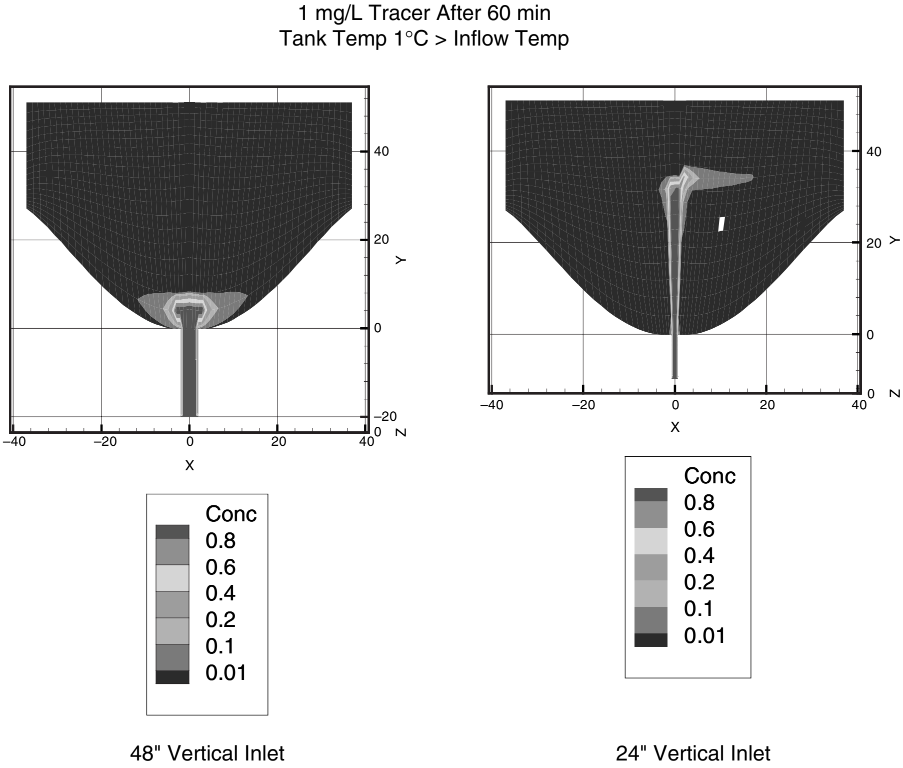
- Elevated tank—1 MG capacity with 48-in.-diameter vertical inlet at the bottom of the tank.
- Standpipe—4 MG capacity with 36-in.-diameter horizontal inlet near the bottom of the tank.

The City of Virginia Beach has another type of tank, namely ground tank. CFD modeling was not performed for this type of tank, primarily to save computational time, but temperature measurements were performed to assist with the evaluation of mixing characteristics for this type of tank.

Elevated Tank

The City of Virginia Beach distribution system has five elevated tanks, with capacities ranging from 0.25 MG to 1 MG. The inlet diameters range from 16 to 48 in. (Tank B). Historical data indicated that the area of the distribution system supplied primarily by Tank B lost chlorine residual faster than other areas of the distribution system, especially during the summer months.

The effect of mixing patterns for Tank B (with 48-in.-diameter inlet) as well as for other elevated tanks (with 16-in.-diameter inlet) illustrated using outputs from the CFD model (Figure 1). SCADA data indicated that the average inflow rates into the elevated tanks were approximately 1,000 gpm during the summer. Figure 1 shows the mixing pattern after 1 hr of fill time at an average inflow rate of 1,000 gpm for 16-in.-diameter and 48-in.-diameter vertical inlets. The temperature of the stored water is assumed to be 1°C higher than the inflow because during the summer the water temperature in the tank is warmer than the



48" Vertical Inlet

24" Vertical Inlet

Figure 1 Effect of inlet momentum on mixing characteristics

water in the buried pipes. For the 48-in.-diameter inlet, where fluid velocity, and thus momentum, are much lower, the mixing zone is confined to the area surrounding the inlet. For the 16-in.-diameter inlet, the increased fluid velocity and momentum extend the mixing zone nearly to the water surface in the tank within the same fill duration.

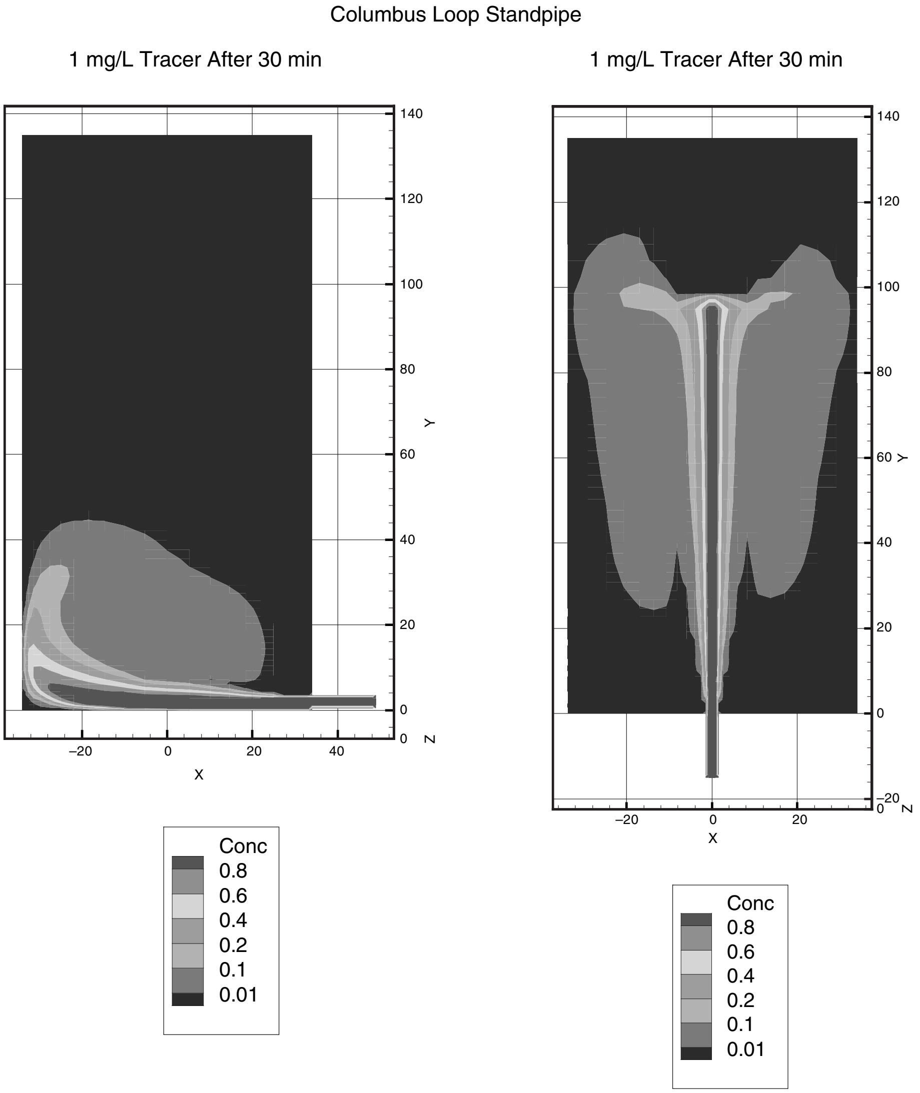
The CFD modeling results predict that mixing of the bulk content of Tank B (over the entire tank depth) is not achieved. The SCADA data indicated that the daily volume turnover in Tank B is approximately 32%. Thus, the water above the mixing zone is occasionally supplied into the distribution system from the tank, causing poor water quality.

Standpipe

The City of Virginia Beach distribution system also has four standpipes, with capacities ranging from 2 MG to 4 MG. These standpipes are tall compared with the diameter, and water enters the bottom of the tanks in the horizontal direction. The inlet diameters range from 24 to 36-in., and the average inflow rates were approximately 2,000 gpm.

The effect of inlet orientation on mixing characteristics is demonstrated by an output from the CFD model for Tank C (standpipe) under isothermal conditions (Figure 2). Figure 2 shows the mixing pattern after 30 minutes of fill time at an average inflow rate of 2,000 gpm for a 36-in. horizontal inlet (existing configuration) versus a 36-in. vertical inlet (assumed for illustration purposes). When the inlet orientation is horizontal, the path of the jet is in the direction of minimum water length. The water jet hits the far wall of the tank within 15 minutes and is thus unable to mix the stored water in the top portion of the tank. However, when the inlet orientation is vertical, the path of the jet is in the direction of maximum water length. In this case, there is sufficient water length in the direction of flow for mixing to occur without significant loss of momentum due to impact with the tank walls. In this case, mixing extends to the upper regions of the water column. It should be noted here that if the tank water temperature is warmer than the inflow temperature (which may occur during the summer), the mixing region may not always extend to the upper regions of the water column for the tank geometry and inlet configuration illustrated.

All the standpipes in the Virginia Beach distribution system have horizontal inlets near the bottom of the standpipes. The distribution system areas in the vicinity of these standpipes generally maintain a chlorine residual. The SCADA data indicated that the water turnover in these standpipes is less than 20%, implying that the water supplied to the distribution system by the standpipes comes from the bottom portions of the standpipes. Because fresh water comes in and out of the common inlet/outlet pipes near the bottom of the standpipe, the water quality appears to be good. However, when there is a fire flow or main break or a very high demand, water of poor quality from the top portions of the standpipes will also be drawn into the distribution system. In such cases, water of poor quality will enter the distribution system from the standpipes because the water in the top portions of the standpipes is stagnant and not well mixed. The City of Virginia Beach recently converted from free chlorine to chloramines for secondary disinfection. The presence of stagnant chloraminated water in the top portions of the tank can cause nitrification and loss of residual.



Columbus Loop Standpipe

1 mg/L Tracer After 30 min

1 mg/L Tracer After 30 min

36" Horizontal Inlet
Isothermal Condition (20°C)

36" Vertical Inlet
Isothermal Condition (20°C)

Figure 2 Effect of inlet pipe orientation on mixing characteristics

Temperature Effects

Temperature differences between the inflow and the bulk water in a tank will affect mixing characteristics in the tank. During the summer, water temperature in a tank will be warmer than that of water entering the tank because piping buried in the ground is cooler than the ambient temperature. The cooler, denser, inflow will tend to remain at the bottom of the tank. With the inlet at the tank bottom, the inflow's lower temperature and inadequate inlet momentum can combine to cause poor mixing and result in thermal stratification of water in a tank.

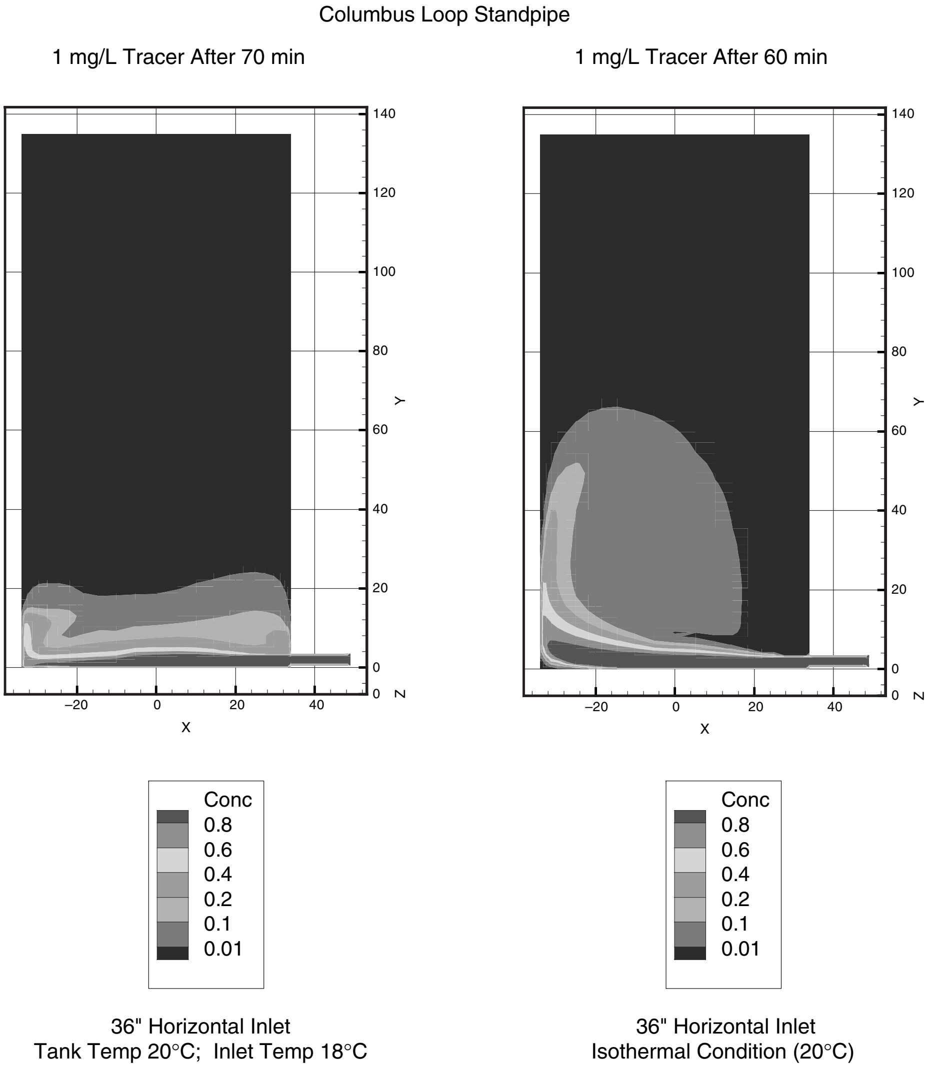
The effect of temperature difference between inflow and stored water on mixing characteristics in a tank is shown in Figures 3 and 4. Figure 3 illustrates the mixing characteristics for a horizontal inlet while Figure 4 illustrates the mixing characteristics for a vertical inlet, both inlets being of the same diameter. The CFD modeling results indicate that the mixing zone for the isothermal condition (when the stored water and inflow are at the same temperature) is greater than the negative buoyancy condition (when the stored water is warmer than the inflow). During the summer, if the stored water is heated during the day, a negative buoyancy condition may exist. Therefore, it is anticipated that during the summer, mixing characteristics may be less than desirable, resulting in increased potential for stratification.

EVALUATION OF MIXING CHARACTERISTICS WITH TEMPERATURE MEASUREMENTS

Poor mixing conditions in the tanks lead to thermal stratification of the water inside the tanks. The three types of tanks that were evaluated with CFD modeling (ground tank, elevated tank, and standpipe) were also selected for field testing through the use of temperature probes to identify any possible thermal stratification.

- Ground tank (5 MG capacity)
- Elevated tank (1 MG capacity)
- Standpipe (2 MG capacity)

Based on the previous CFD modeling results, the ground tank is expected to have good mixing characteristics and no thermal stratification; the elevated tank and standpipe are expected to have inadequate mixing characteristics and a greater degree of thermal stratification. The study objective was to assess if stratification conditions could be identified in the selected tanks by the vertical temperature profile. Thermistors were placed in the tanks at several fixed depths, and hourly temperature monitoring was conducted at these water heights for 2 to 3 weeks.



Columbus Loop Standpipe

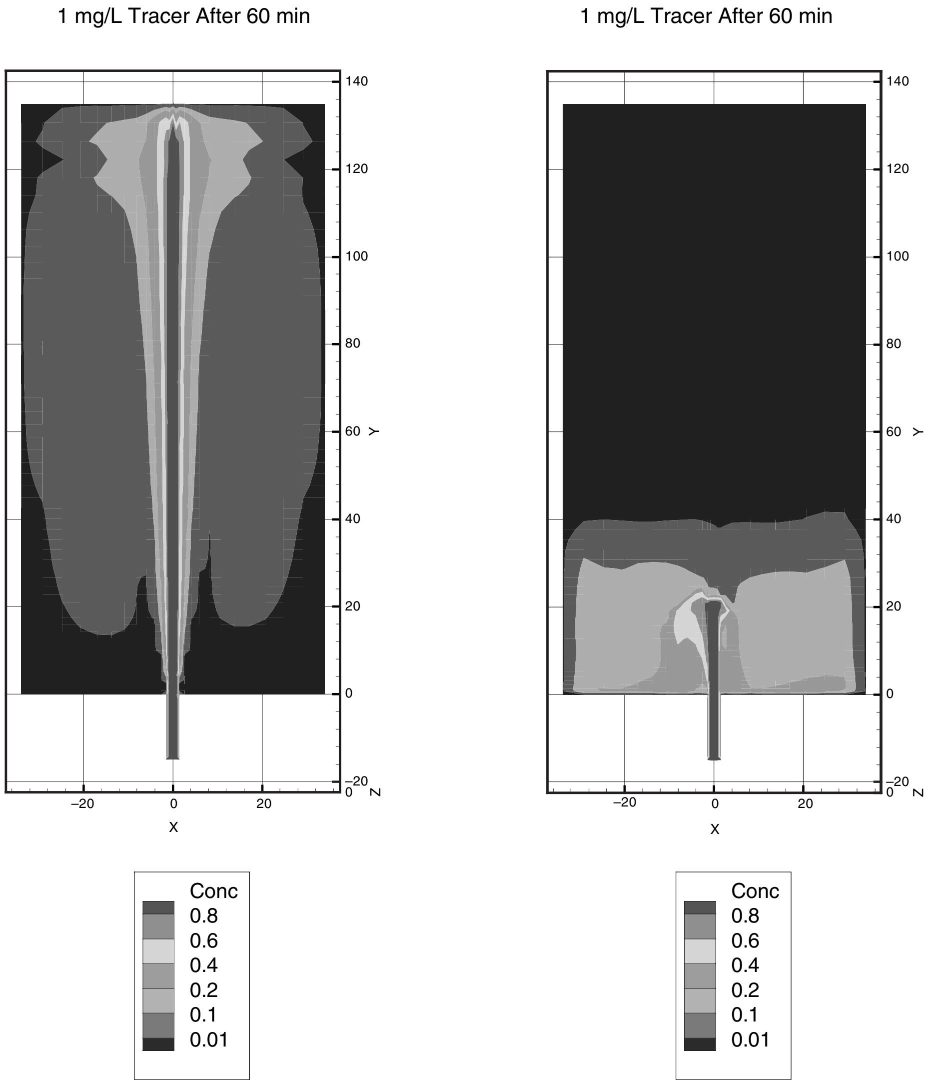
1 mg/L Tracer After 70 min

1 mg/L Tracer After 60 min

36" Horizontal Inlet
Tank Temp 20°C; Inlet Temp 18°C

36" Horizontal Inlet
Isothermal Condition (20°C)

Figure 3 Effect of temperature difference between stored water and inflow on mixing characteristics for horizontal inlet



1 mg/L Tracer After 60 min

1 mg/L Tracer After 60 min

36" Vertical Inlet
Isothermal Condition (20°C)

36" Vertical Inlet
Tank Temp 1°C > Inflow Temp

Figure 4 Effect of temperature difference between stored water and inflow on mixing characteristics for vertical inlet

Ground Tank

Fixed temperature probes were placed 2, 15, 28, and 43 ft above the tank bottom, and a floating probe was set 2 ft below the water surface. Temperatures recorded over a 16-day period (June 6–22, 2000) in the ground tank are presented in Figure 5.

The overall gradual increase in temperature is due to the progressive increase of the ambient temperature and thus progressive warming of the city's water supply during the testing period. As the average outside air temperature increases, the temperatures at the top and bottom of the tank begin to diverge at about noon, and the water appears partially stratified by the evening. The maximum temperature difference each day between water at the top and bottom of the tank was approximately 0.8°F, the top of the tank being at the higher temperature. Water was usually withdrawn from the tank for approximately 7 hr each day, between approximately 1 p.m. and 8 p.m. It is evident from Figure 5 that the drain cycles do not promote mixing and prevent stratification. This result is expected because there is no jetting action within the tank during the drain period.

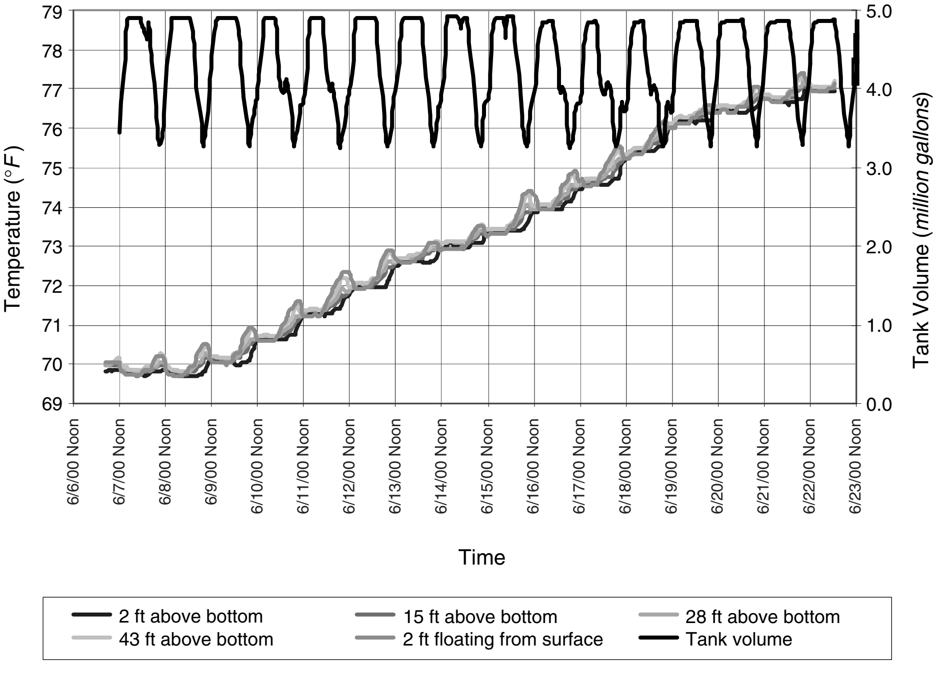


Figure 5 Temperature profile at various depths for ground tank

Although the temperature profiles exhibit the onset of stratification each day, the temperatures over the entire depth of the tank appear to converge during the night, when the tank is refilled. The temperature over the entire depth of the tank is nearly uniform by approximately 1 a.m. each day. This convergence of temperature implies that the water is well mixed in the tank. The tank is typically filled for about 5 hr daily, between approximately 8 p.m. and 1 a.m. This observed behavior provides strong evidence of the importance of the jetting action induced during the fill cycle for achieving well-mixed conditions. The field temperature measurements confirm CFD modeling analyses, which indicate that this ground tank has favorable mixing characteristics. Thermal stratification in this tank is negligible, as expected.

Elevated Tank

Fixed temperature probes were placed at 7, 12, and 19 ft above the tank bottom, and a floating probe was placed 2 ft below the surface. The results of the temperature monitoring data over a 20-day period (June 22–July 12, 2000) for this elevated tank are presented in Figure 6.

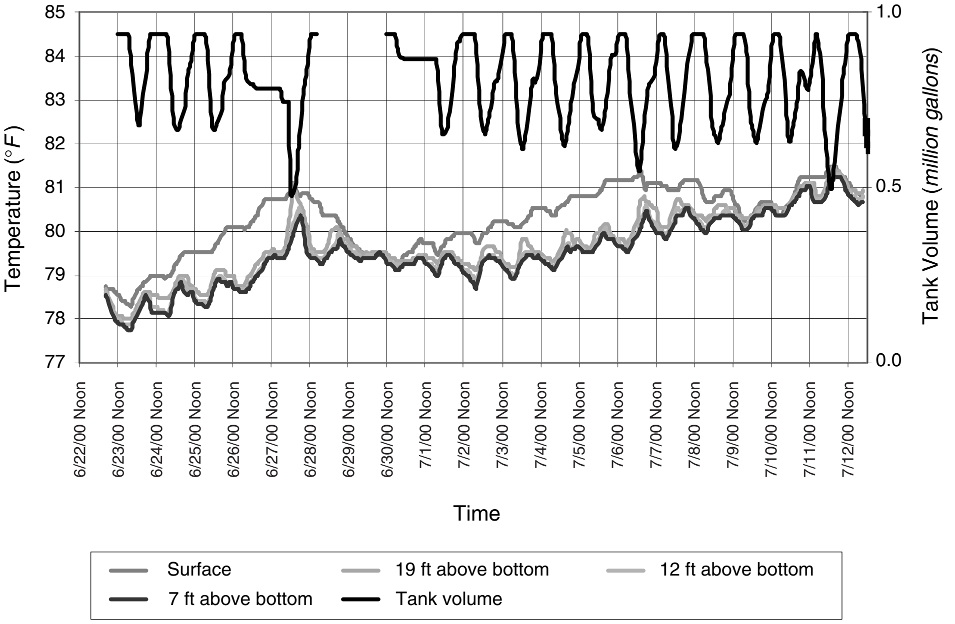


Figure 6 Temperature profile at various depths for elevated tank

The following observations can be made from Figure 6:

- A cyclical variation of water temperatures within the tank appears to trend with the fill-and-draw operation. During the fill cycle, the cooler inflow tends to lower the temperature between the bottom of the tank and the 19-ft depth. During the drain cycle, a trend of increasing temperature is observed as the warmer water in the upper region of the tank is drawn down and comes into contact with the temperature probes.
- There is minimal tendency of the water temperatures to converge (from top to bottom) during the filling cycles. This result implies that only limited mixing is achieved throughout the full depth of the water during a typical tank-filling cycle. In fact, with the exception of two separate periods (June 29 and July 9–10) during the 20-day monitoring, temperature convergence between the water surface and the lower depths does not occur.
- The temperature difference between the probes set 7 ft and 19 ft above the bottom is small, but the temperature at 19 ft from the bottom is consistently higher than that at 7 ft from the bottom. The temperature difference between the 19-ft depth and the water surface is consistently relatively high (about 1°F).

These temperature observations confirm the CFD modeling results, which indicated that mixing of the bulk content of the tank (over the entire tank depth) was not achieved. This observation is especially applicable for the top 15 ft of water, between the water surface (about 34 ft from the bottom) and the depth at 19 ft from the bottom. Generally, this elevated tank appears to exhibit long-term stratification. The observed temperature stratification in this tank appears to be primarily due to the large diameter (4 ft) of the inlet/outlet riser, which results in low inlet momentum.

Standpipe

For this standpipe, the fixed temperature probes were placed at 20, 50, and 130 ft above the bottom, and the floating probe was set 2 ft below the water surface. The results of the temperature monitoring data over a 14-day period (September 1–14, 2000) are presented in Figure 7.

The following observation can be made from Figure 7:

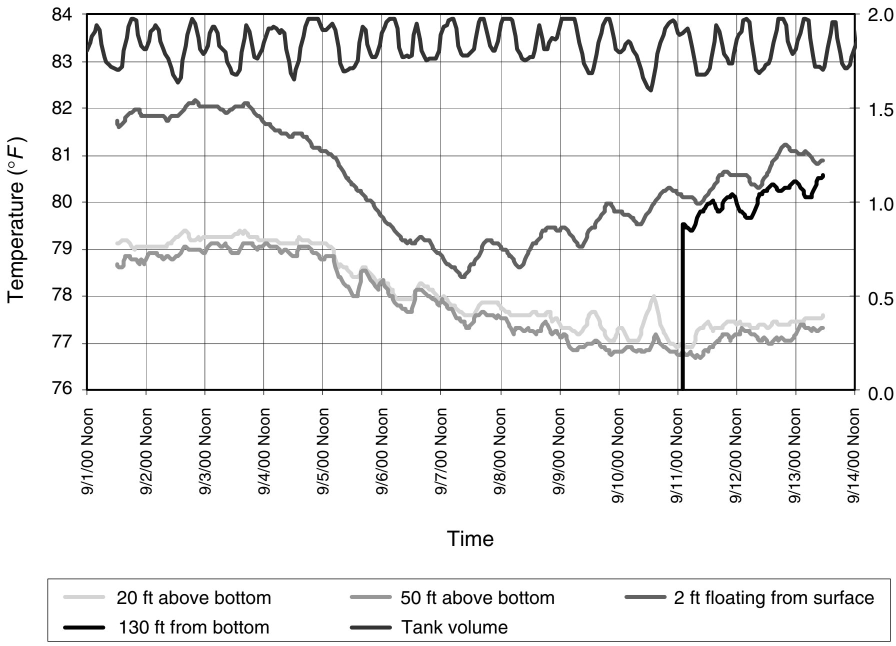


Figure 7 Temperature profile at various depths for standpipe

- The temperature differential between the water at 50 and 20 ft from the bottom was insignificant (generally less than 0.5°F). These data imply that the water at these depths was well mixed.
- The temperature of the surface layer (2 ft below the surface) was consistently higher than the temperature measured at lower depths. The temperature differential between the water at 2 ft below the surface and at 50 ft from the bottom generally ranged from approximately 1° to 3°F.
- This large temperature difference indicates that stratification occurred at the upper region of the tank (above 50 ft).
- The temperature 2 ft below the surface of the tank did not converge with the temperature 50 ft from the bottom of the tank during the fill cycles. This result implies that the fill cycle did not mix the water throughout the tank. The decrease in temperature near the surface of the tank between September 4 and 7 was due to a decrease in ambient temperature.

These observations indicate that mixing over the entire depth of the tank was not achieved. Long-term stratification was observed due to lack of mixing and was particularly evident for water above 50 ft. The results for this tank confirm the CFD modeling analysis, which predicted poor mixing characteristics and thus high potential for stratification. As mentioned previously, the horizontal orientation of the inlet for this tank does not promote good mixing. In this configuration, when the jet hits the vertical wall, momentum is lost and the extent of the mixing region is reduced.

CHLORINE RESIDUAL MEASUREMENTS

When water in any portion of a storage tank is not mixed adequately, the water age in that portion of the tank increases with the resulting loss of residual for chlorinated systems and loss of residual and nitrification for chloraminated systems. The City of Virginia Beach staff measured chlorine near the top and bottom of most of their storage tanks. The results are presented in Table 1 below:

Table 1 Chlorine residuals in storage tanks

Date	Capacity (*MG*)	Type	Cl_2 Top (*mg/L*)	Cl_2 Bottom (*mg/L*)
09/12/00	5.0	Ground	**1.0**	**1.0**
09/12/00	1.0	Elevated	0.6	0.8
09/12/00	1.0	Elevated	1.0	1.7
09/12/00	1.0	Elevated	0.9	1.2
	Average for elevated tanks		**0.8**	**1.2**
09/13/00	4.0	Standpipe	0.2	1.0
09/13/00	2.0	Standpipe	0.3	1.3
09/12/00	2.0	Standpipe	0.0	0.2
09/12/00	4.0	Standpipe	0.0	1.0
	Average for standpipes		**0.1**	**0.9**

The chlorine residual data indicate that the mixing characteristics are good in the ground tank and poor in the standpipes and some of the elevated tanks. These results confirm the findings from CFD modeling and temperature measurements.

RECOMMENDED DESIGN MODIFICATIONS FOR IMPROVED MIXING IN TANKS

In order to achieve good mixing characteristics in storage tanks, the key design parameters are adequate inlet momentum and suitable inlet pipe orientation, and the key operational parameters are adequate volume turnover and fill time. If operational changes are not sufficient to improve mixing or cannot be made due to hydraulic or other constraints, design changes should be considered.

The key design parameters of inlet pipe diameter and orientation should produce an inlet momentum adequate to completely mix the water in the tank within the fill time. For example, it was found that the standpipes in the city's distribution system have an average fill time of approximately 3 hr and an average inflow rate of approximately 2,000 gpm. Figure 8 shows that an inlet pipe diameter of 24 in. at 2,000 gpm inflow cannot provide adequate momentum to mix the bulk water within 3 hr of fill time. The CFD modeling results for this situation illustrate that the top third of the tank still remained relatively unmixed after 3 hr. When the inlet diameter was reduced to 12 in., the entire tank was well mixed by 1 hr of filling, as shown in Figure 9. The inlet momentum for the 12-in. inlet was 30 ft^4/sec^2.

For the City of Virginia Beach's standpipes, the general design recommendations for improving the mixing and turnover characteristics were:

- Change orientation of the inlet pipe from horizontal to vertical to direct the water jet through the maximum length of water in the tank.
- Reduce the inlet pipe diameter for each standpipe so that an inlet momentum of approximately 20–30 ft^4/sec^2 is achieved (this inlet momentum is adequate to mix the bulk water within the fill time).

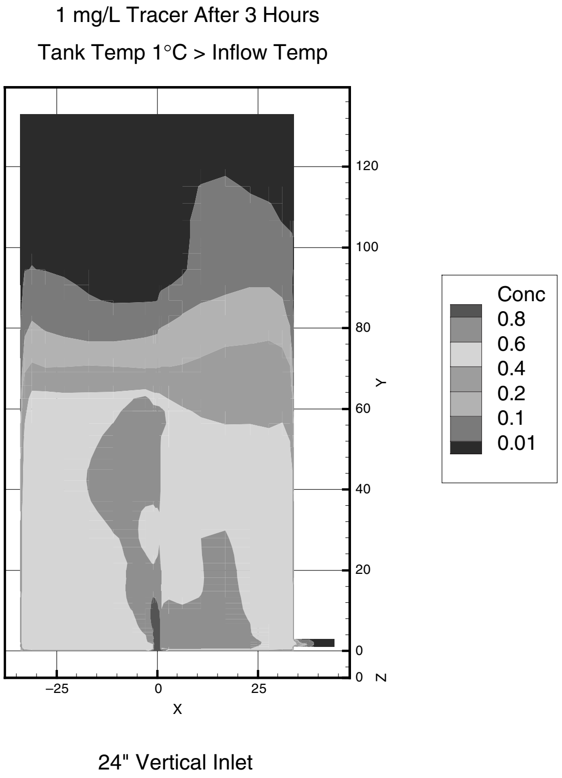


Figure 8 Mixing characteristics after 3 hours' inflow for 24-in.-diameter inlet

- Move the inlet pipe near the center of each standpipe to minimize the effect of the wall on the water jet and allow the maximum amount of stored water to be entrained in the jet.

The general design recommendations for improving the mixing and turnover characteristics of the city's elevated tanks were:

- Reduce the diameter of the inlet pipe in the tank with large-diameter inlet so that an inlet momentum of approximately 20–30 ft^4/sec^2 is achieved.
- Remove any baffle or other flat plates near the inlet pipes so that inlet momentum is not lost when the inlet water jet hits the plate.

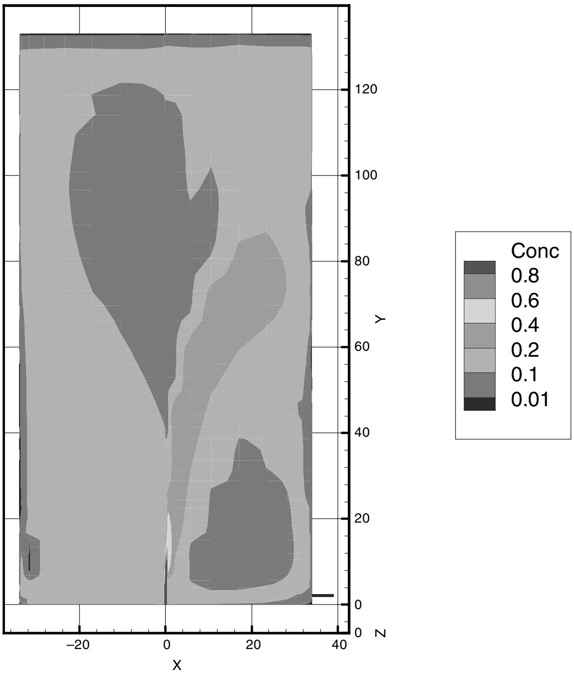


Figure 9 Mixing characteristics after 1 hour inflow for 12-in.-diameter inlet

RESULTS OF MODIFICATIONS

At the time of this writing, piping modifications were made at one of the standpipes. To determine the benefit of the tank piping modifications on its mixing characteristics, water temperatures were measured at various depths within this standpipe as well as another standpipe whose inlet pipe had not been modified. The two standpipes are physically essentially identical, with the exception of their inlet pipes. The unmodified tank had the original 24-in.-diameter horizontal common inlet/outlet pipe, while the modified tank had a 12-in.-diameter vertical inlet pipe located in the center of the tank (fitted with a duckbill-style rubber check valve) and a 24-in.-diameter horizontal outlet pipe.

To assess the variation of temperature with depth in the tanks, the temperature probes were placed at approximately 5, 40, 75, and 140 ft below the maximum water surface in both tanks. The probe at 5 ft below the water surface had a float attached to it so it always remained at 5 ft below the water surface. Before and after each installation of the probes, their calibration was checked. Appropriate adjustments were made to the raw temperature data to account for any calibration drift that was identified. Water temperature was monitored at the modified tank from July 10 to 26, 2001, and at the unmodified tank from August 1 to 5, 2001.

Results for Modified Standpipe

Figure 10 shows the results of the temperature monitoring for the modified standpipe and a plot of the recorded water level in the tank.

The following observations can be made from this figure:

- The water temperatures at 5 ft, 40 ft, and 140 ft below the maximum water surface were consistently very similar. The maximum temperature difference between the top portion of the tank (at 5 ft and 40 ft from the water surface) and the bottom portion of the tank (140 ft from the water surface) was less than 1°F, and lasted for a few hours only. It appears that the water in the tank typically began to thermally stratify as the water level dropped and the temperature rose each day, but then was mixed during the tank-refilling cycle each evening.

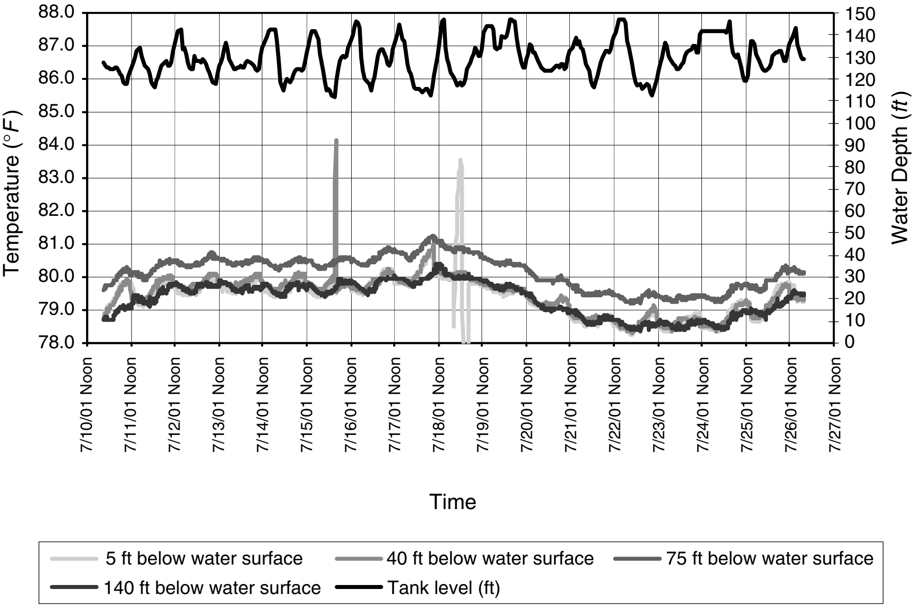


Figure 10 Temperature profile at various depths for modified standpipe

- There is an anomaly with the data recorded by the probe located 75 ft from the water surface, where the recorded water temperature was consistently approximately 1°F higher than the water in the rest of the tank. This may be due to a measurement error by the temperature probe that was not apparent in the calibration check.
- The temperature at 5 ft below the maximum water surface reached 84°F on July 15 and the temperatures at both 5 ft and 40 ft below the maximum water surface reached 83°F on July 18. These results were due to the tank water level dropping below the normal range and thus exposing the temperature probes set at 5 ft and 40 ft below maximum water surface to the ambient air temperature. Because the tank was heated during the day, the ambient air temperature was higher than the water temperature.

The temperature data indicate that although stratification typically begins to occur each day, as seen by the increasing difference in recorded temperature between the top (5- and 40-ft depth) and bottom (140-ft depth) of the tank, the temperatures at the top and bottom of the tank converge during the night, when the tank is refilled. This convergence of the temperatures indicates that water in the tank is adequately mixed.

Results for Unmodified Standpipe

Figure 11 shows the results of the temperature monitoring for the unmodified standpipe and a plot of the recorded water level in the tank.

The following observations can be made from this figure:

- The temperature difference between the probes set at 5 ft and 40 ft below the maximum water surface was minimal (generally less than 0.5°F), and the temperatures at these two depths were consistently warmer than the temperatures at the two lower depths.
- The water temperature at 40 ft below the water surface was consistently higher than that at 75 ft below surface (generally more than 1°F). The temperature of the water at 75 ft below the surface was consistently higher than that at 140 ft below the surface (generally more than 2°F). The progressive increase in water temperature from the bottom of the tank to 40 ft below the water surface indicates that much of the tank was thermally

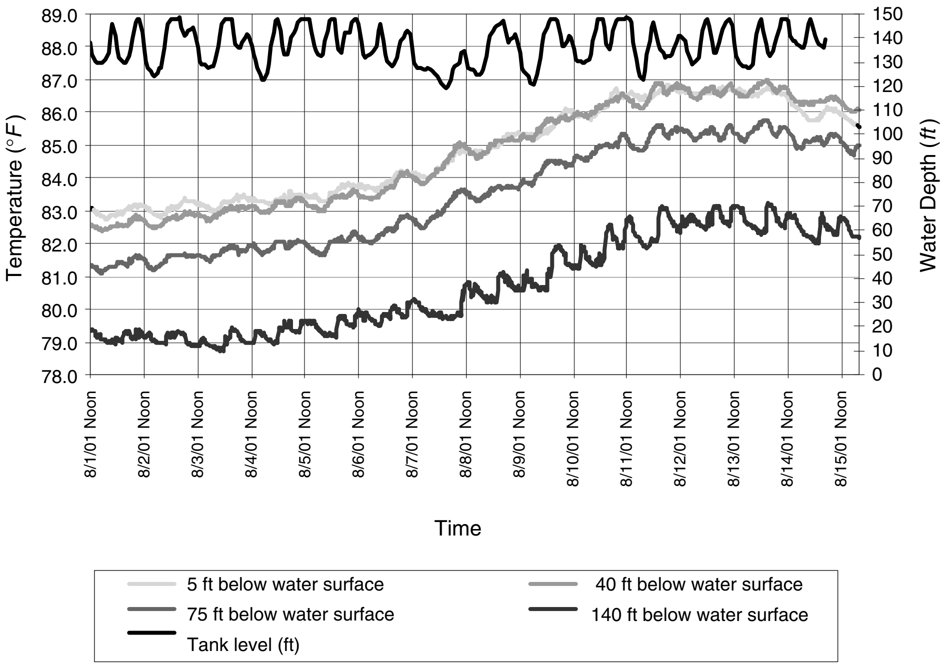


Figure 11 Temperature profile at various depths for unmodified standpipe

stratified. The difference in water temperature between the bottom and top of the tank was consistently between 3.5° and 4°F.

- The temperatures at the different depths showed very little convergence during the tank-filling cycles. In particular, as the tank refilled each evening and the water temperature at the bottom of the tank decreased, there usually was no corresponding decrease in water temperature at the top of the tank. This implies that the fill cycle failed to mix the water throughout the full depth of the tank.
- The overall gradual increase in water temperature appears to be due to a combination of the progressive increase of the temperature of water at the top of the standpipe and an increase in the temperature of water entering the standpipe from the distribution system during the testing period.

Comparison of Mixing Characteristics for the Modified and Unmodified Standpipes

The temperature data indicate that the inlet piping modifications at one of the standpipes were successful in preventing temperature stratification during a period when a similar unmodified standpipe was stratified. During the temperature monitoring period, the average daily water level fluctuation in the modified tank was approximately 25 ft. The average daily water level fluctuation in the unmodified tank was approximately 21 ft. The number of cycles per day was higher for the unmodified tank than for the modified tank. There was often both a midmorning to noon and an evening refilling period for the unmodified tank. Considering both the magnitude of the daily level fluctuations and the frequency of fluctuations, the overall volume turnover in both tanks was similar. However, the key difference between the two tanks that explains the temperature stratification observed in the unmodified tank is the piping modifications made to the modified tank C1. The significant difference in the water temperature profiles at various depths between the two tanks was primarily due to the inlet/outlet pipe modifications at the modified tank. The pipe modifications in this tank have improved mixing conditions throughout the tank, and the level of mixing appears to be adequate to prevent temperature stratification.

CONCLUSIONS

Inlet momentum is a key factor for mixing of water in storage tanks. The higher the inlet momentum, the better the mixing characteristics in the storage tanks. Increasing the flow rate could be a simple way to increase momentum, but it may not be practical to increase the flow rate into tanks because of limitations of system hydraulics. For example, a pump may not be available at the tank location and the distribution system pressure may not be high enough to get desirable increases in flow rates, or even if a pump were available, it may not be possible to increase the pumping rate into the tanks. In such cases it may be more feasible to increase the inlet momentum by increasing the velocity with a reduced inlet diameter.

The location and orientation of the inlet pipe relative to the tank walls have a significant impact on mixing characteristics. For example, when the height of a tank is much larger than the diameter or width, the location of the inlet pipe in the horizontal direction at the bottom of the tank will likely result in incomplete mixing of the water in the tank. For this type of tank, horizontal orientation of the inlet pipe could result in the water jet hitting the vertical wall of the tank before sufficient time has elapsed for mixing, thus resulting in loss of inlet momentum. The mixing characteristics of a tank also depend on the initial water depth in the tank. When the inlet orientation is horizontal and the initial water height is very high, the inlet momentum may not be able to mix all the water in the tank completely during the fill cycle. As the water enters the bottom of the tank, the momentum can dissipate completely before reaching the top of the tank. In such a case, the top portion of the tank remains unmixed.

Temperature differences between the bulk tank water and the inflow have a significant effect on the mixing characteristics of a storage tank. Even temperature differences of less than 1°C can affect mixing characteristics. Depending on the location of the inlet pipe and tank geometry, the water entering the tank from the buried pipes may be cooler than the bulk water in the tank during the summer or warmer than the bulk water in the tank during the winter. Temperature differential may result in density gradients inside the tank and cause stratification and poor mixing.

Computational fluid dynamic modeling and temperature measurements can be effective tools to predict mixing characteristics in storage tanks. The adequacy of inlet momentum and suitability of inlet pipe orientation to provide sufficient mixing throughout the storage tank, and the impact of design changes on mixing characteristics can be effectively investigated using CFD modeling. Temperature measurements of storage tanks are relatively simple to conduct and are highly effective in predicting the potential for thermal stratification that results from poor mixing characteristics.

CHAPTER · 41

Extracting Meaning From Distribution System Data

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BACKGROUND

Among the many factors contributing to ever-increasing quantities of distribution system water quality data being collected by U.S. water utilities is a growing regulatory emphasis on distribution system water quality in general. Regardless of the system-specific reasons—whether voluntary or compliance-related—few would argue with the perception that utilities will collect more distribution system water quality data in the future than in years past. The common practice of plotting trends and applying simple statistics to distribution system parameters, such as heterotrophic plate counts (HPCs), is not particularly useful for decision making at water systems that deal with large quantities of data collected from numerous sites under dynamic environmental conditions. More complex statistical tools offer insights and provide an analytical framework that can simplify decision making.

A one-year monitoring program was undertaken to evaluate HPCs and other select parameters biweekly at 26 sites selected outside of the system's bacteriological compliance sites. The 26 special sites were in presumed low-flow areas or near pressure zone boundaries, or were otherwise expected to represent suboptimal distribution system water quality conditions. A statistical approach was used to examine the overall value of the special monitoring program. For example, statistics were used to discern whether the data from the HPC values at special sites were different from similar data collected at compliance sites. Several other data relationships were also probed. The nature of HPC data makes them particularly subject to analytical results that are difficult to handle statistically. For example, a typical laboratory result is TNTC (too numerous to count). Such a result, if ignored or if set to zero for purposes of reporting, would improperly bias the evaluation. In addition, HPC data, like many environmental parameters, may not be normally distributed, which precludes the use of the many common statistical tests.

In this case study, the authors applied statistical tools that allowed for non-normal data distributions, and they developed techniques for appropriately analyzing censored HPC data. Use of databases also enabled consideration of vast quantities of historical data. By using these and other techniques, most of the historical data could be retained, allowing the decisions to be made on the basis of a much larger data set. Although this project focused on just a few parameters, the data management procedures and statistical techniques used in this case study can be applied to most types of distribution system water quality data to make practical operational decisions. This chapter includes specific steps for maximizing the statistical value of data collected for compliance purposes. It also provides guidance for designing and implementing statistically valid voluntary monitoring programs. Finally, this chapter provides advice on handling non-normal or censored data and guidelines for avoiding common errors in statistical analyses.

DISTRIBUTION SYSTEM WATER QUALITY MONITORING PRACTICES

Effective distribution system monitoring programs are designed to achieve the following objectives:

- Ensure that drinking water meets applicable regulatory requirements
- Quantify changes in water quality as water travels through the system
- Provide early warning of emergencies or problematic events that threaten water quality

There are two recognized types of monitoring. *Compliance monitoring* is systematic sampling of the distribution system to demonstrate the continued delivery of safe drinking water to all service areas [1, 2]. Compliance monitoring programs encompass sampling requirements from various drinking water regulations. *Special-purpose monitoring* is designed to provide information about distribution system characteristics and, if necessary, locate problematic areas [1, 2]. Special-purpose programs are generally more complex than compliance programs and may require special knowledge. Examples of such programs include monitoring to locate the origin of a contamination event or locate a coliform biofilm. For large systems, Geldreich [1] emphasizes the importance of understanding flow patterns and retention times in the network when designing a monitoring program.

Compliance Monitoring

Promulgation of the Total Trihalomethane (TTHM) Rule in 1979, followed by the Total Coliform Rule (TCR) and the Surface Water Treatment Rule (SWTR) in 1989, signaled a new regulatory emphasis on water quality in the distribution system. This emphasis continued with the promulgation of the Lead and Copper Rule (LCR) in 1991 and the Stage 1 Disinfectants–Disinfection By-products Rule (Stage 1 D–DBPR) in 1998.

New regulations stemming from the 1996 Amendments to the Safe Drinking Water Act (SDWA) will further emphasize the role of the distribution system in delivering safe water. For example, the Initial Distribution System Evaluation (IDSE), as included in the negotiated Stage 2 Microbial–Disinfection By-products Agreement in Principle, will refine distribution system sampling requirements. Federal cross-connection control regulations are also under consideration.

Monitoring programs (Table 1) are critical in determining compliance with rules that regulate water quality in the distribution system. Regulatory requirements generally dictate water quality parameters that must be monitored, associated laboratory methodologies, sample siting, sampling frequency, and reporting requirements. In addition to satisfying federal requirements for distribution system water quality monitoring, programs implemented by utilities should ensure that the samples accurately characterize water quality in all service areas. In the interest of public health, this objective must be given higher priority than collecting a minimum number of samples as specified by regulations [1].

Special-Purpose Monitoring

Some utilities choose to supplement required analyses with additional parameters to gain more insight into distribution system water quality. In addition to the required parameters of disinfectant residual and pH, the parameters most often monitored by water systems are heterotrophic plate count (HPC), temperature, turbidity, and color [3].

Water systems that use chloramines to maintain a disinfectant residual often undertake additional distribution system monitoring to detect early indications of nitrification. Approximately 66% of systems

Table 1 Regulatory requirements for distribution system monitoring

Parameter	Sample Location	Regulatory Limit	Rule	Comments
• Total coliforms • Fecal coliforms	Distribution system entry point	• 5% samples positive for total coliform bacteria • Repeat samples positive for fecal coliform or *E. coli* bacteria	TCR	Number of samples required is determined by the population served
Disinfectant residual	Distribution system entry point	Minimum 0.2 mg/L on a continuous basis	SWTR	Applies to systems using surface water supplies
Disinfectant residual or heterotrophic plate count (HPC)	Throughout distribution system	Detectable disinfectant residual or HPC ≤ 500 cfu/mL in 95% of samples collected each month for two consecutive months	SWTR	Applies to systems using surface water supplies
Disinfectant residual	Distribution system	• MRDL chlorine 4.0 mg/L • MRDL chloramine 4.0 mg/L, running annual average	Stage 1 D–DBPR	Surface water systems serving >10,000 people
Total trihalomethanes (TTHMs)	Throughout distribution system	80 µg/L, running annual average based on quarterly samples	Stage 1 D–DBPR	Surface water systems serving >10,000 people

Table continued next page.

Table 1 Regulatory requirements for distribution system monitoring *(continued)*

Parameter	Sample Location	Regulatory Limit	Rule	Comments
Sum of five haloacetic acids (HAA_5)	Throughout distribution system	60 μg/L, running annual average based on quarterly samples	Stage 1 D–DBPR	Surface water systems serving >10,000 people
• Lead • Copper	At customers' taps	• Lead action level: 0.015 mg/L at 90% • Copper action level: 1.3 mg/L at 90%	LCR	Number of samples determined by population served
pH	Representative points in distribution system	Minimum of 7.0	LCR	

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that use chloramines experience nitrification, and 25% are affected by moderate to severe nitrification [4]. Negative consequences of nitrification include loss of disinfectant residual, increased concentrations of nitrate and nitrite, increased HPCs, and decreased alkalinity, pH, and dissolved oxygen (DO) [4–6]. An occurrence of nitrification may require considerable effort and expenditure to mitigate. In a survey of 67 chloraminating utilities, 40% routinely monitored for nitrification [4]. All of the utilities monitored bacteriological indicators and chloramine residual. The majority of respondents also monitored ammonia, nitrite, nitrate, pH, and temperature. DO was monitored less frequently, but Wilczak et al. [4] suggested that it may be a good indicator of nitrification, particularly during warm water (>15°C) conditions.

RATIONALE FOR APPLYING STATISTICAL ANALYSES TO DISTRIBUTION SYSTEM WATER QUALITY

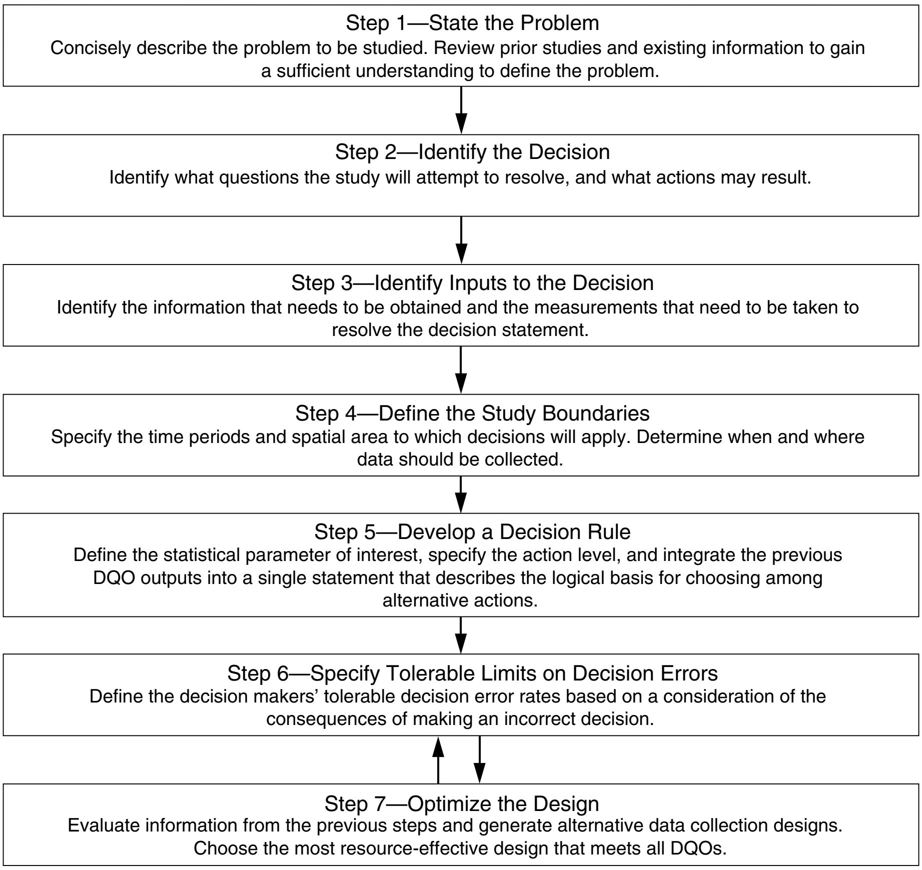
Distribution system monitoring programs, whether conducted for compliance or other reasons, can quickly generate large quantities of data. For example, monitoring five parameters at 20 sites on a biweekly basis generates over 2,500 data points per year.

Analyzing data derived from distribution system monitoring programs can be a complicated task, particularly since data are collected under dynamic conditions from multiple sampling locations. Fortunately, statistical methods are powerful tools in the quest to understand what distribution system data mean. Appropriate application of statistical tools can lead to improved understanding of spatial water quality trends, temporal water quality trends, and interrelationships among multiple water quality parameters. Statistical methods can also provide the ability to predict changes in water quality parameter(s) resulting from changes in other parameters or operational practices. Additionally, results of statistical analyses can provide a quantitative basis for making operational decisions.

DESIGNING STATISTICALLY VALID DISTRIBUTION SYSTEM MONITORING PROGRAMS

As previously discussed, monitoring requirements for compliance programs are generally straightforward and are often dictated by water quality regulations. When statistics will be used for data analysis and decision making, however, the monitoring program should be designed with this specific goal in mind.

When designing a sampling program, investigators need to address multiple issues, such as the study problem or objectives, water quality parameters of interest, number of sample sites required, identification of sample sites, establishment of a sample schedule, analytical method, and statistical methods or other methods for data analysis. The USEPA's *Guidance for the Data Quality Objectives Process* [7] provides a systematic approach for addressing these issues and developing programs to collect data for decision-making purposes. The seven steps (Figure 1) of the Data Quality Objectives (DQO) process guide the user in completing the following tasks [7]:



Source: Guidance for the Data Quality Objectives Process *[7]*

Figure 1 Seven steps of the DQO process

- Defining the study problem statement or objectives
- Identifying the type(s) of data to collect
- Identifying the appropriate conditions for data collection
- Specifying tolerable limits on decision errors

Note that the actual data collection program design is completed in the final step of the DQO process. The outputs from each of the preceding steps, as described below, are critical in designing an effective sampling program.

Step 1: State the Problem

Like the scientific method, the DQO process begins with developing a statement of the question or problem to be investigated. USEPA [7] recommends developing a concise statement that includes background information relevant to the study objective. Components of the problem statement may include [7]:

- Conditions prompting the study (e.g., nitrification in the distribution system or frequent occurrences of total coliforms)
- Current understanding of the problem (i.e., regulatory context, stakeholders, political issues, results of previous studies, constraints on sampling design)
- Literature information about similar studies, sampling methods, analytical methods, etc.

Step 2: Identify the Decision

During this phase of the DQO process, the investigators seek to identify the "principal study question" and possible courses of action to resolve the problem. The principal study question should clarify unknown conditions and identify the information needed to solve the problem. The principal study question also serves as the basis for determining the action required to solve the problem under investigation [7]. Examples of principal study questions include:

- Do lead concentrations in distribution system samples exceed the action level?
- Are HPCs in perceived "worst-case" areas of the distribution system higher than HPCs at TCR compliance monitoring sites?

After developing the principal study question, the next step involves identification of possible actions, including the "no action required" alternative. Thereafter, the principal study questions and action alternatives are incorporated into a single "decision statement" of the following form [7]:

> Determine whether or not [*unknown environmental conditions/issues/criteria from the principal study question*] require (or support) [*taking alternative actions*].

Investigators should be prepared for the possibility that multiple decision statements will need to be formulated and resolved to address a single problem.

Step 3: Identify Inputs to the Decision

The objective of this step is to determine the informational inputs required to resolve the decision statement (identified in step 2) and determine environmental variables that must be measured. Questions such as the following should guide the identification of the informational needs.

- Is information about water main sizes needed?
- Are water treatment plant finished water quality data needed?

Investigators should consider whether monitoring, modeling, or some combination of these techniques will most effectively provide the required data. In some instances, existing site-specific data, such as compliance monitoring data, or published data may satisfy informational needs.

During this step in the DQO process, the investigators should also consider the available analytical and measurement methods. USEPA [7] recommends listing potential methods along with the detection limits and limits of quantitation.

Step 4: Define the Study Boundaries

The fourth step in the DQO process involves defining the spatial and temporal boundaries of the investigation. This activity first focuses on developing a clear understanding of the population(s) of interest. Examples of populations include disinfectant residual concentrations in small-diameter water mains and copper concentrations at customers' taps. Once the populations of interest are understood, the investigators can establish appropriate geographic boundaries (i.e., pressure zones, certain main sizes, etc.) to investigate the problem identified in the decision statement. Additionally, the investigators must define any temporal boundaries on the decision and consider environmental factors to determine when to collect data. A critical action in defining study boundaries is identifying practical constraints on data collection. For example, in distribution system sampling, investigators should seek commercial, rather than residential, sample sites to facilitate access. Finally, this step involves defining the "scale of decision making," or the "smallest, most appropriate subsets of the population (sub-populations) for which decisions will be made based on the spatial or temporal boundaries" [7].

Step 5: Develop a Decision Rule

This DQO step helps to further clarify key aspects of the study: the statistical parameter(s) of interest, the action level(s) with which the parameter(s) will be compared, the scale of decision making, and the action alternatives. The data collection program is designed to *estimate* the statistical parameter(s) of interest (e.g., mean, median, percentile, etc.) for the population(s) under study. Once the decision maker knows the estimated statistical parameter(s), he/she can compare the estimated population parameter with a fixed action level to decide among the alternative actions. An example of an action level is fixed regulatory standards, such as a minimum required disinfectant residual or regulatory limit on total TTHMs. USEPA [7] recommends developing a decision statement of the form:

> If the [*parameter of interest*] within the [*scale of decision making*] is greater than [*action level*], then implement [*alternative action A*]; otherwise implement [*alternative action B*].

Step 6: Specify Tolerable Limits on Decision Errors

Application of statistical methods for decision making always requires consideration of decision errors. Since our approach involves estimating characteristics of a population based upon the characteristics of a sample, errors in sampling design or analytical measurements can lead to incorrect conclusions. Therefore, statisticians seek to control decision error by setting tolerable limits.

Controlling decision errors involves identifying a *null hypothesis* (H_0) and an *alternative hypothesis* (H_A) for each decision. The null hypothesis is assumed to be true in the absence of compelling data that indicate otherwise. Conversely, the alternative hypothesis is assumed to be false in the absence of compelling data to the contrary. A *false positive*, or Type I, decision error results when the null hypothesis is incorrectly rejected in favor of the alternative hypothesis. The investigator sets the tolerable probability of committing a false positive decision error by establishing the significance level, a, for statistical testing. A *false negative*, or Type II, decision error occurs when the null hypothesis is accepted, although it is false. Further details about controlling decision errors are provided by USEPA [7].

Step 7: Optimize the Design

In the seventh and final DQO step, the outputs from the first six steps are incorporated into an optimized sampling design. This step involves review of existing environmental data and developing alternative designs for collecting new data. Technical sampling design techniques are discussed in detail elsewhere [7, 8].

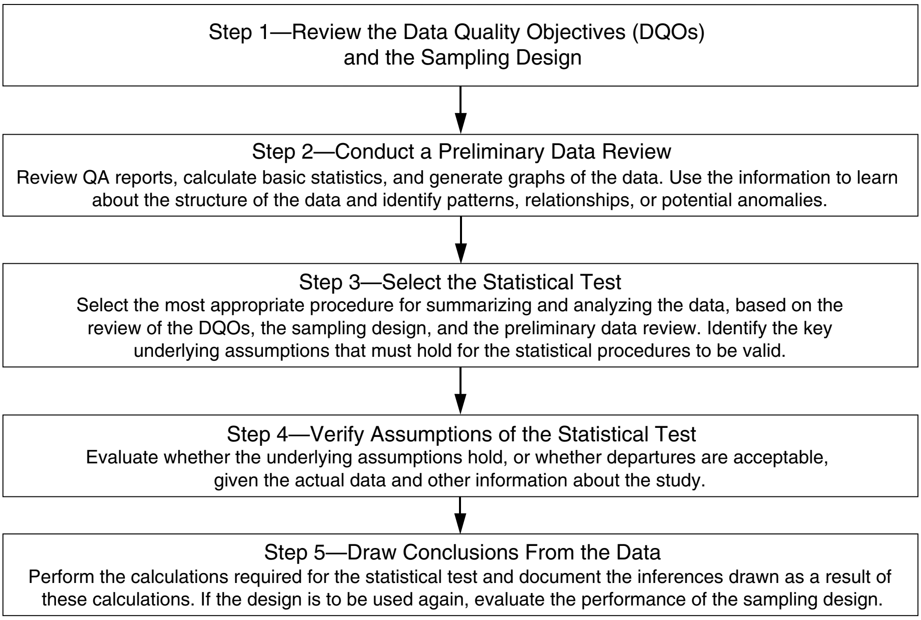
USEPA guidance [7] also includes specific steps to optimize the number of samples given the data requirements of the planned statistical analysis and available resources. Finally, step 7 involves documenting the data collection design and underlying assumptions in a sampling and analysis plan.

OVERVIEW OF STATISTICAL METHODS

After planning activities, such as those described in the DQO process, are completed, the next steps involve data collection and assessment. During the data assessment phase, investigators seek to draw conclusions about the environmental conditions of the population from which the sample was taken. For example, investigators may use HPC data from a discrete number of sample sites to draw conclusions about microbiological water quality throughout the distribution system.

USEPA offers guidance for statistical analysis of environmental data in *Guidance for Data Quality Assessment: Practical Methods for Data Analysis* [9]. The Data Quality Assessment (DQA) process complements the DQO process described in the previous section. While utilities are not required to use DQO or DQA for voluntary investigations of distribution system water quality, these tools are presented here because they offer sound guidance on planning statistically valid monitoring programs and applying statistical tools to analyze data and guide decision making.

The steps of the DQA process (Figure 2), along with descriptions of various statistical tools, are described in further detail in the following



Step 1—Review the Data Quality Objectives (DQOs) and the Sampling Design

Step 2—Conduct a Preliminary Data Review
Review QA reports, calculate basic statistics, and generate graphs of the data. Use the information to learn about the structure of the data and identify patterns, relationships, or potential anomalies.

Step 3—Select the Statistical Test
Select the most appropriate procedure for summarizing and analyzing the data, based on the review of the DQOs, the sampling design, and the preliminary data review. Identify the key underlying assumptions that must hold for the statistical procedures to be valid.

Step 4—Verify Assumptions of the Statistical Test
Evaluate whether the underlying assumptions hold, or whether departures are acceptable, given the actual data and other information about the study.

Step 5—Draw Conclusions From the Data
Perform the calculations required for the statistical test and document the inferences drawn as a result of these calculations. If the design is to be used again, evaluate the performance of the sampling design.

Source: Guidance for the Data Quality Assessment: Practical Methods for Data Analysis *[9]*

Figure 2 Five steps of the DQA process

sections. Examples are also included to illustrate the application and usefulness of a statistical approach for understanding the data.

Step 1: Review DQOs and the Sampling Design

USEPA [9] recommends reviewing the outputs from the DQO process before analyzing the data to ensure that all the investigators understand the study objectives; assumptions underlying the data collection design; and methods for data collection, measurement, and reporting. If investigators seek to apply the DQA process to data that were collected without DQO planning, this step involves developing DQOs to establish criteria for evaluating the data quality and completing the data analysis [9].

Step 2: Conduct a Preliminary Data Review

This DQA step includes activities to aid investigators in developing an understanding of the data set. Typically, the data analysis team examines field and laboratory quality assurance reports for indications of anomalies or problems in the data collection effort. The preliminary data review also utilizes basic statistical calculations and graphical techniques to provide the investigators with insight into the data structure and trends or patterns.

At this stage of the DQA process, the investigators should begin to recognize and consider any special issues associated with the data set. For example, many environmental data are not normally distributed. Since many statistical tests assume data are normally distributed, tests that rely upon this assumption may not be applicable to the data set. Therefore, during this DQA step, investigators should seek to identify the data distribution (e.g., normal distribution, log-normal distribution, etc.). If the distribution is unknown, investigators should be prepared to utilize special, *nonparametric* statistical techniques. Nonparametric tests are based on ordered rank values rather than on individual data points, and can be used when the distribution is non-normal or unknown.

Censoring is another special issue associated with many environmental data sets. Censored data sets include some data points for which actual measured values are not available. Censored values may be reported as “not detected” (ND) or “below detection limit” (BDL). For HPC data, censoring may occur in the form of measurements reported as “too numerous to count” (TNTC). If the data are censored, investigators should determine how to handle these data during the preliminary data review. For example, HPC data are often center-censored. Measurements reported as TNTC or >300 colony-forming units/milliliter (cfu/mL) can be set to 300 cfu/mL. Measurements reported as <1 cfu/mL can be set to 0 cfu/mL or 0.5 cfu/mL. When substituting values in censored data sets, it is advisable to order and rank the data and apply nonparametric methods to analyze the ranks instead of the measurement values.

Statistical methods for summarizing basic information about data (Table 2), commonly referred to as *summary statistics* or *descriptive statistics*, may include the number of observations, measures of central tendency, measures of dispersion, measures of relative standing, measures of distribution symmetry or shape, and measures of association. Table 3 illustrates the use of summary statistics for a censored HPC data set.

Graphical techniques are also important tools in the preliminary data review step of DQA. Data plots can quickly reveal patterns and trends that may not be discerned through statistical calculations alone. They may also provide a quick, visual indication of whether assumptions, such as normally distributed data, are valid. Some of the most common graphical methods for data—histograms, box-and-whisker plots, scatter plots, and probability plots—are briefly described below. For in-depth

Table 2 Examples of summary statistics

Statistic	Definition
Number of observations (count)	Quantity of measurements in the data set (n)
Measures of Central Tendency	
Mean	The mathematical “average” of a data set, calculated by summing the measurements and dividing the sum by the number of observations, n
Median	The middle observation when the measurements are ordered (i.e., the 50th percentile value)
Mode	The measurement value that occurs most frequently in the data set
Measures of Dispersion	
Variance	A measure of dispersion calculated by (1) determining the distance (i.e., difference) between each measurement and the sample mean, (2) squaring each distance, (3) summing the squares of the distances, and (4) dividing the sum by n–1. The distances are squared to avoid the potential of positive and negative distances for canceling out.
Standard deviation	The positive square root of the variance
Range	The difference between the largest and smallest measurements in a data set
Interquartile range	The difference between the upper quartile (i.e., 75th percentile) and the lower quartile (i.e., 25th percentile)

Table continued next page.

Table 2 Examples of summary statistics *(continued)*

Statistic	Definition
Measures of Relative Standing	
Percentiles	For an ordered data set, the *p*th percentile is the measurement below which *p* percent of the data fall. For example, if a measurement of 10 is the 90th percentile, 90% of the data in the distribution are less than 10.
Measures of Distribution Symmetry or Shape	
Skewness	Dimensionless measure of symmetry
Measures of Association	
Pearson's Correlation Coefficient	• Measure used with a scatter plot to detect *linear* correlation between two variables • Ranges between –1 (perfect negative correlation) and +1 (perfect positive correlation); value of 0 indicates variables are not correlated
Spearman's Rank Correlation Coefficient	Nonparametric complement of Pearson's Correlation Coefficient, calculated by replacing data values with ranks and applying the Pearson's Correlation Coefficient formulas

Table 3 Summary statistics for HPC (cfu/mL) data set

	Disinfectant	
Parameter	Chloramine	Chlorine
Count	1,039	3,037
Mean	7.84	14.56
Median	0	2
Geometric mean	3.82	6.21
Harmonic mean	2.02	2.91
Standard deviation	35.78	45.61
90th percentile	8	26
Minimum	0	0
Maximum	300	300

discussion of these and numerous other graphical methods, refer to USEPA's *Guidance for the Data Quality Assessment: Practical Methods for Data Analysis* [9] or a statistical text [10, 11].

Histograms

Histograms are constructed by dividing the data range into constant intervals, then assigning each data point to the appropriate interval. The data range and intervals are displayed on the x-axis, and the number of data points per interval is displayed on the y-axis. A bar is used to depict the number of data points in each interval. An example histogram from the utility case study is provided in Figure 3.

Histograms provide a visual indication of symmetry and skewness of the data. A quick glance at the histogram in Figure 3 confirms that these data are asymmetric, or *skewed*—that is, they are not symmetric around a central point. Furthermore, the data are *skewed to the right*, as indicated by the long tail of high HPC values.

Box-and-whisker plots

Box-and-whisker plots, such as those in Figure 4, provide considerable information about the range and variability of the data. The box itself encloses the middle 50% of the data. The upper and lower box values

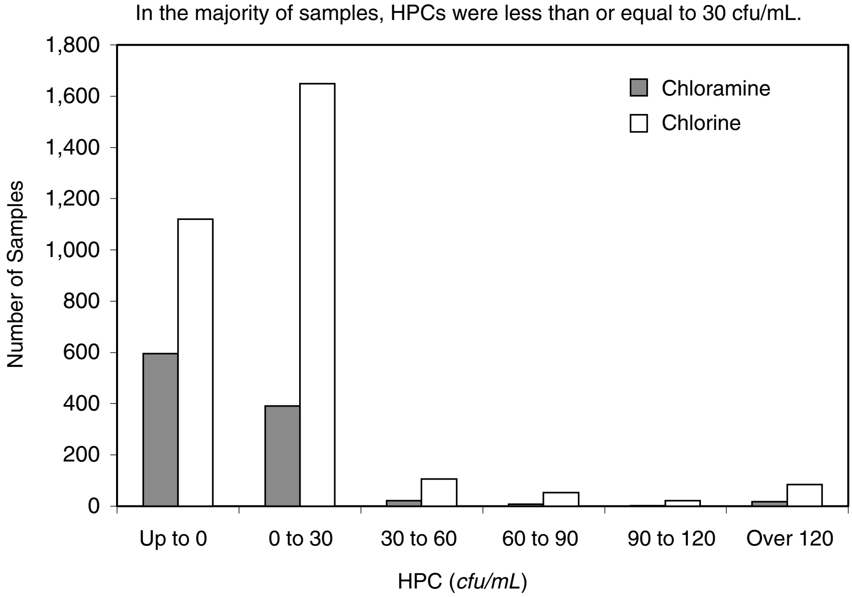


Figure 3 Histogram of HPC results

indicate the 75th and 25th percentiles, respectively, and the line within the box represents the median (50th percentile). The whiskers above and below the box provide information about the data symmetry, and extreme data are shown as individual data points.

Scatter plots

Scatter plots are useful for examining relationships between two continuous variables. Paired observations (x, y) are plotted as individual data points on scatter plots. When there is no relationship between the variables, the data points appear randomly scattered, as in the example in Figure 5. However, when the variables are related, the relationship is depicted as a function (i.e., $y = f[x]$) on the scatter plot. For example, a linear trend would appear on a scatter plot for variables with a linear correlation.

Probability plots

Probability plots are examples of quantile-quantile plots. A quantile plot is constructed with values ranging from 0.0 to 1.0 on the x-axis, representing the fraction of data points. The y-axis includes the data

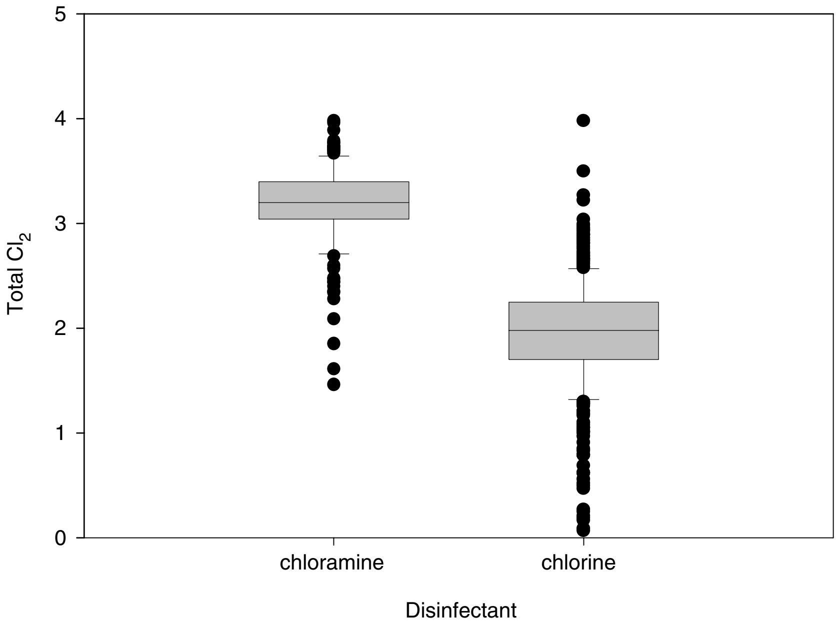


Figure 4 Comparison of total chlorine concentrations with different disinfectant residuals

values. For each point on the plot, x = the fraction of points exceeded and y = the data value. For example, if the first quartile (i.e., the 25th percentile) has a value of 5, and the third quartile (i.e., the 75th percentile) has a value of 15, two points on the quantile plot would include (0.25, 5) and (0.75, 15). Quantile-quantile plots are created by modifying the quantile plot procedure to plot the quantiles of two data sets against one another. That is, the x-axis includes data values from one data set, and the y-axis includes data values from another set. The x and y coordinates of each point plot represent equal quantiles within the respective data sets.

Probability plots are quantile-quantile plots in which data sets are plotted against known distributions. Common distributions include the normal distribution, log-normal distribution, and uniform distribution. The shape of the curve on a probability plot provides investigators with a visual indication of whether the data can be modeled with the distribution in question. For example, data that appear linear on probability plots may follow the distribution represented on the x-axis. Even if data do not

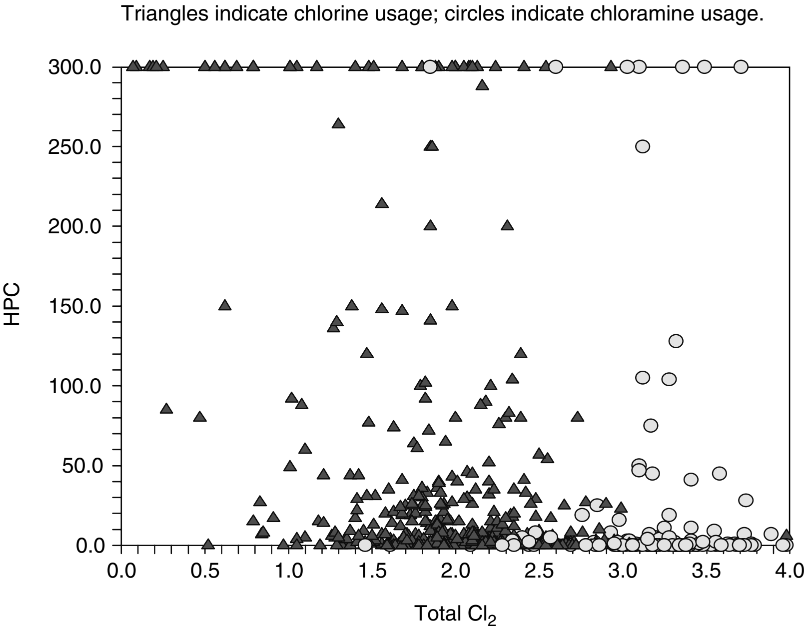


Figure 5 Lack of strong relationship between HPC (*cfu/mL*) and total chlorine (*mg/L*)

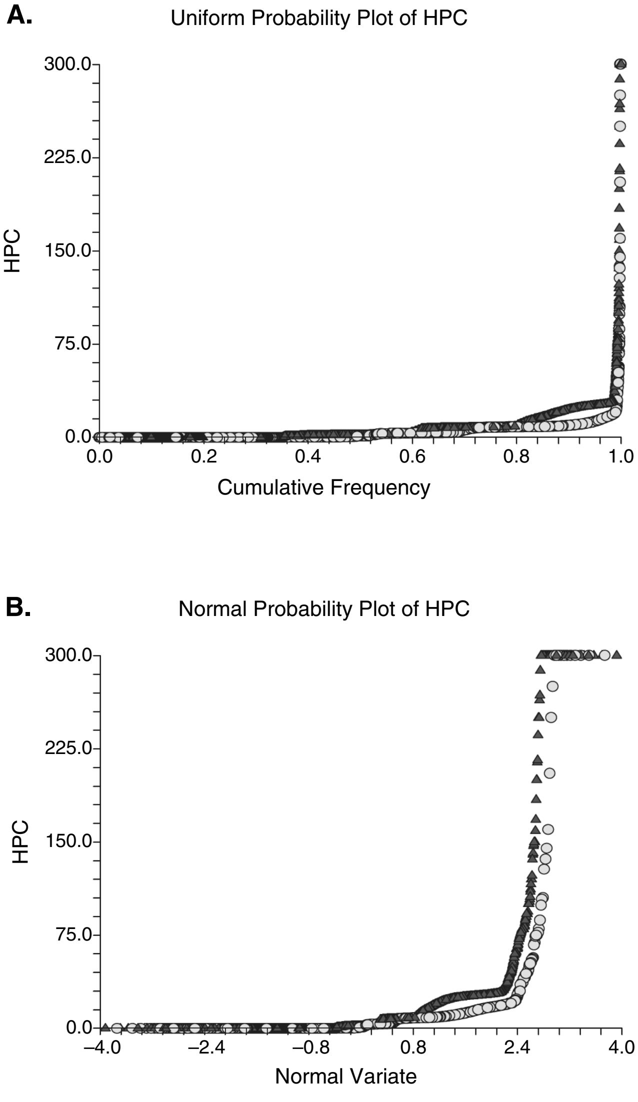
appear linear on the probability plot, the shape of the curve may provide some insight about the distribution shape and skewness.

Figure 6 includes probability plots for the uniform and normal distributions. As seen in this example, the HPC data do not conform to either of these distributions, since neither of the curves is linear. However, the shapes of the curves depict the right skewness of the data.

Step 3: Select the Statistical Test

This step of the DQA process includes selection of potential statistical tests to meet the data objectives. Activities include:

- Reviewing the data objectives
- Defining the hypotheses to test (i.e., null hypothesis and alternative hypothesis) and the false positive (α) and
- False negative (β) error rates
- Identifying potential statistical tests



Triangles represent chlorine data, and circles represent chloramine data. The uniform probability plot illustrates the extreme right skewness of the data. Equally spaced data would have plotted as a straight line, and data shaped as a bell curve would have plotted with an S shape on the uniform cumulative probability plot. The normal probability plot illustrates that the data are not normally distributed. Normal data would have plotted as a straight line. The impact of the TNTC observations set at 300 can also be seen. The chlorine data are usually above the chloramine data, also indicating that the HPC levels with chlorine are higher.

Figure 6 Probability plots of HPC under the different disinfectant regimes

- Noting underlying assumptions and limitations for the tests.

General categories of hypothesis testing—tests for single populations and tests comparing two populations—are discussed in the following sections. Table 5 [9] summarizes specific statistical methods that are useful for understanding environmental data and the application, assumptions, limitations, and verifications for these methods. Equations and procedures for implementing the statistical tests are provided elsewhere [9–11].

Tests for single populations

Tests for single populations, or *one-sample* tests, are designed to compare a population parameter—such as the mean, median, or a percentile—with a fixed action level or threshold value. Recall that the goal of sampling is to collect appropriate data for *estimating* the population parameter under consideration. Proper application of one-sample tests requires that the population parameter be compared with a fixed value (e.g., a regulatory level), not an estimated value (e.g., a background concentration). For example, a one-sample test would be suitable for determining whether 95% of HPC measurements are below a fixed level, such as 100 cfu/mL. However, it would not be appropriate for determining whether total iron concentrations are higher in corroded areas of the distribution system than in noncorroded areas.

One-sample tests that compare a population mean with a fixed value include the One-Sample t-Test, the Wilcoxon Signed Rank (One-Sample) Test for the Mean, and the Chen Test. These tests are designed to test the following hypotheses [9]:

- Case 1: H_0: $\mu \leq C$ vs. H_A: $\mu > C$
- Case 2: H_0: $\mu \geq C$ vs. H_A: $\mu < C$
- Case 3: H_0: $\mu = C$ vs. H_A: $\mu \neq C$

Where:

μ = population mean
C = threshold value

The One-Sample Proportion Test is another test for comparing a parameter from a population with a fixed threshold value. This test can be used to test a proportion (i.e., the fraction of a population) or percentile (i.e., the percentage of the population with a value less than a given value). The applicable hypotheses [9] for these tests are summarized in Table 4.

Variations of two tests already discussed—the One-Sample Proportion Test and the Wilcoxon Signed Rank (One-Sample) Test for the Mean—allow investigators to compare the median of a population with a fixed value. When applied to median testing, the One-Sample Proportion Test is referred to as the Sign Test for a Median. Since the population median

Table 4 Hypotheses for one-sample proportion test

	Hypotheses for Proportion Tests	Hypotheses for Percentile Tests
Case 1	H_0: $P \leq P_0$ vs. H_A: $P > P_0$	H_0: $100P^{th} \geq C$ vs. H_A: $100P^{th} < C$
Case 2	H_0: $P \geq P_0$ vs. H_A: $P < P_0$	H_0: $100P^{th} \leq C$ vs. H_A: $100P^{th} > C$
Case 3	H_0: $P = P_0$ vs. H_A: $P \neq P_0$	H_0: $100P^{th} = C$ vs. H_A: $100P^{th} \neq C$

P = proportion of the population ($0 \leq P \leq 1$)
P_0 = fixed proportion ($0 \leq P_0 \leq 1$)
C = threshold value

is the 50th percentile of the population, the Sign Test is conducted by setting P to 0.5 and following the One-Sample Proportion Test Procedure. For a symmetric distribution, where the population median is equivalent to the population mean, the Wilcoxon Signed Rank Test provides a comparison of the median with a fixed value. Medians comparing medians with threshold values can be used to test the following hypotheses [9]:

- Case 1: H_0: $\mu \leq C$ vs. H_A: $\mu > C$
- Case 2: H_0: $\mu \geq C$ vs. H_A: $\mu < C$
- Case 3: H_0: $\mu = C$ vs. H_A: $\mu \neq C$

Where:

μ = population median
C = threshold value

Tests for comparing two populations

In addition to tests for single populations, the statistical toolbox includes tests for comparing two populations with one another or with fixed threshold values. For example, the tests discussed in this section might be appropriate for comparing disinfection by-product (DBP) concentrations at sites with low residence times with DBP concentrations at sites with high residence times. These tests could also be used to gauge the effects of operational changes, such as changes in metal concentrations after implementation of a corrosion control program. Note, however, that if samples for both populations (i.e., “before” treatment population and "after" treatment population) are collected from the same sample sites, the data are not independent and require special consideration [9].

Parametric tests exist for comparing the means, proportions, or percentiles—including medians—of two independent populations. Student's Two-Sample t-Test (Equal Variances) and Satterthwaite's Two-Sample t-Test (Unequal Variances) compare population means. The following hypotheses apply to these tests [9]:

- Case 1: H_0: $\mu_A \geq \mu_B$ vs. H_A: $\mu_A < \mu_B$
- Case 2: H_0: $\mu_A \leq \mu_B$ vs. H_A: $\mu_A > \mu_B$
- Case 3: H_0: $\mu_A \leq \mu_B$ vs. H_A: $\mu_A \neq \mu_B$

The Two-Sample Test for Proportions, which applies for proportions or percentiles, directly compares the proportions of two populations that share a specific characteristic. For example, this test could be used to compare the proportion or percentage of samples testing positive for total coliforms with the proportion or percentage of positive total coliform samples recorded by a neighboring water system. Hypotheses for the Two-Sample Test for Proportions may take one of the following forms [9]:

- Case 1: H_0: $P_1 \geq P_2$ vs. H_A: $P_1 < P_2$
- Case 2: H_0: $P_1 \leq P_2$ vs. H_A: $P_1 > P_2$
- Case 3: H_0: $P_1 = P_2$ vs. H_A: $P_1 \neq P_2$

Nonparametric versions of these tests, including the Wilcoxon Rank Sum Test and the Quantile Test, compare the shape of the two populations, rather than the values themselves. According to USEPA [9], hypotheses for these tests generally take the form:

- H_0: The distributions of Population A and Population B are identical (no difference).
- H_A: Part of the distribution of Population A is located to the right of the distribution of Population B (Population A is different from Population B).

USEPA [9] notes that the structure of the hypotheses is important, and Population A is usually the area of interest while Population B is the reference area.

Table 5 Statistical hypothesis tests

Statistical Test	Purpose	Assumptions	Limitations	Verification Tests
		Hypothesis Tests for a Single Population		
		Tests for a Mean		
One-Sample t-Test	To test hypotheses about the mean of the population from which a random sample was selected	• Random sample (i.e., data are independent) • Sample mean is approximately normally distributed (if distribution is unknown) Central Limit Theorem states that the sample mean will be approximately normal for large sample sizes	• Limited application if sample includes outliers because outliers exert strong influence on sample mean and standard deviation • Limited application for censored data (i.e., below detection limit values) since method tests the mean rather than rank or proportion	• Test for independence • Test for normality if sample size is small • Test for outliers
Wilcoxon Signed Rank (One-Sample) Test for the Mean	To test hypotheses about the mean of the population from which the sample was selected	• Random sample (i.e., data are independent)	• Limited application if many of the data values are the same because this test relies upon the *ranks* of the data values	• Test for independence

Table continued next page.

Table 5 Statistical hypothesis tests *(continued)*

Statistical Test	Purpose	Assumptions	Limitations	Verification Tests
Chen Test	To compare the population mean from a right-skewed sample with a fixed value (e.g., regulatory level, threshold level)	• Random sample (i.e., data are independent) • Right-skewed distribution (common for environmental data)	• Limited application if there are a large number of censored data, but may be able to substitute a value for nondetects (e.g., half the detection limit)	• Test for independence • Examine data (e.g., construct histogram) and calculate skewness to confirm right-skewness
Tests for a Proportion or Percentile				
One-Sample Proportion Test	To test hypotheses about a proportion or percentile of the population from which the sample was selected	• Random sample (i.e., data are independent)	• Robust to outliers, but confirm that outliers are not data errors	• Test for independence or confirm that sampling was random
Tests for a Median				
Sign Test for a Median	To test hypotheses about the median of the population from which the sample was selected	Note: This test is the same as the One-Sample Proportion Test with test percentile set to 0.5 (median). See the assumptions, limitations, and verification tests for the One-Sample Proportion Test.		

Table continued next page.

Table 5 Statistical hypothesis tests *(continued)*

Statistical Test	Purpose	Assumptions	Limitations	Verification Tests
Wilcoxon Signed Rank (One-Sample) Test for the Median	To test hypotheses about the median of the population from which the sample was selected	Note: This test is the same as the Wilcoxon Signed Rank (One-Sample) Test for the Mean. See the assumptions, limitations, and verification tests for the Wilcoxon Signed Rank (One-Sample) Test for the Mean. For a symmetric distribution, the mean and median are equal. This test is more powerful than the Sign Test for symmetric distributions.		
		Tests for Comparing Two Populations		
		Tests for Comparing Two Means		
Student's Two-Sample t-Test (Equal Variances)	To compare the means of two populations	• Samples from both populations are random (i.e., data are independent) • Sample means from both populations are approximately normally distributed • Variances of the two populations are approximately equal	• Limited application if sample includes outliers because outliers exert strong influence on sample mean and standard deviation • Robust to violations of normality and equality of variance	• Test for independence or confirm that sampling was random • Test for normality if sample sizes are small • Test to confirm that the variances of the two populations are approximately equal • Test for outliers

Table continued next page.

Table 5 Statistical hypothesis tests *(continued)*

Statistical Test	Purpose	Assumptions	Limitations	Verification Tests
Satterthwaite's Two-Sample t-Test (Unequal Variances)	To compare the means of two populations	• Samples from both populations are random (i.e., data are independent) • Sample means from both populations are approximately normally distributed	• Limited application if sample includes outliers because outliers exert strong influence on sample mean and standard deviation • Robust to violations of normality	• Test for independence or confirm that sampling was random • Test for normality if sample sizes are small • Test for outliers
Comparing Two Proportions or Percentiles (Including Medians)				
Two-Sample Test for Proportions	• To compare the proportions or percentiles of two populations • Note: The medians are comparing the 50th percentiles.	• Samples from both populations are random (i.e., data are independent)	• Robust to outliers, but confirm that outliers are not data errors	• Test for independence or confirm that sampling was random

Table continued next page.

Table 5 Statistical hypothesis tests *(continued)*

Statistical Test	Purpose	Assumptions	Limitations	Verification Tests
		Nonparametric Comparisons of Two Populations		
Wilcoxon Rank Sum Test	• To compare two population distributions • Detects complete shifts in distribution • Often used in conjunction with the Quantile Test	• Samples from both populations are random (i.e., data are independent) • Distributions of both populations have approximately the same shape and dispersion	• Limited application if many of the data values are the same because this test relies upon the ranks of the data values • Partially robust to outliers	• Test for independence or confirm that sampling was random • For large samples, construct histogram to confirm distributions have approximately the same shape
Quantile Test	• To compare two population distributions • Detects partial shifts in distribution • Often used in conjunction with the Wilcoxon Rank Sum Test	• Samples from both populations are random (i.e., data are independent) • Distributions of both populations have approximately the same dispersion (i.e., variance)	• Not robust to outliers	• Test for independence or confirm that sampling was random

Step 4: Verify the Assumptions of the Test

In Step 3 of the DQA process, investigators identify potential statistical tests and the underlying assumptions of the selected tests. In Step 4, the data are examined and tested to determine whether the assumptions are valid. This section provides an overview of tests and guidance for the following assumptions and issues:

- Distribution (e.g., assumption that data follow a normal distribution)
- Outliers
- Dispersion (e.g., assumption that two populations have equal variances)
- Censored data
- Independence

Tests for assumptions of distribution

As evident in Table 5, many statistical tests assume the data are approximately modeled by a normal distribution. Thus, tests for normality are among the most important tools in statistical investigations. In addition to the graphical techniques previously described (i.e., histograms, and probability plots), there are a number of quantitative tests for normality. USEPA's [9] recommendations regarding these tests are summarized in Table 6.

In addition to the tests presented in Table 6, *goodness-of-fit* tests can be applied to determine whether the data fit the normal distribution or some other known distribution. Examples of goodness-of-fit tests include the Chi-square test, Kolmogorov-Smirnov (K-S) statistic, and the Lilliefors K-S test. Details of these tests, as well as those described in Table 6, are provided elsewhere [9–11].

If data do not conform to an assumption about the underlying distribution (e.g., normality), it may be possible to mathematically *transform* the data to a data set that has the desired distribution. For example, many environmental data are log-normally distributed. This distribution is characterized by skewness with a long, right tail. Data that follow this distribution can be transformed to a normal distribution by taking the logarithm or natural logarithm of every data point. The transformed data can then be tested for normality using one of the methods described above. If the transformed data are normal, then tests that assume normality can be used to analyze the data.

Table 6 Statistical tests for normality

Test	Sample Size	Recommendation
Shapiro-Wilk Test (W Tests)	≤ 50	Highly recommended
Filliben's Statistic	≤ 100	Highly recommended
Coefficient of Variation	All sizes	Provides rapid confirmation of non-normality, but does not positively confirm normality
Skewness	> 50	Useful for large sample sizes to indicate symmetry
Kurtosis	> 50	Useful for large sample sizes to measure flatness of a distribution near its center
Studentized Range Test	≤ 1,000	Highly recommended for most data, but does not work well for transformed log-normal data
Geary's Test	> 50	Less powerful than other tests, but useful when statistical tables are not available for other tests

Source: Guidance for the Data Quality Assessment: Practical Methods for Data Analysis *[9]*

Tests for outliers

Since some statistical tests are not robust to outliers, it may be necessary to test for outliers. Outliers are data that are extremely large or small with respect to the rest of the data set. Outliers may be valid data points or errors.

Statistical tests for outliers include the Extreme Value Test, Discordance Test, Rosner's Test, and nonparametric Walsh's Test [9]. These tests do not indicate whether an outlier is erroneous. Rather, they indicate whether an extreme value likely belongs in the population distribution. These tests are used to identify outliers for further investigation. Using scientific judgment, the outliers may be discarded or determined to be valid. USEPA [9] recommends performing the statistical test calculations with and without the questionable data to determine how such data affect the analysis.

Tests for dispersion

When comparing the means of two populations, it may be necessary to determine whether the dispersions of the two populations are approximately equal. If the variances are similar, a Student's Two-Sample t-Test may be suitable. If the variances are not equal, Satterthwaite's Two-Sample t-Test (Unequal Variances) may be appropriate. Tests for determining the equality of variances include the F-Test, Bartlett's Test, and Levene's Test [9].

Guidance for censored data

As previously discussed, data sets that include points reported as "not detected" or above a threshold (e.g., too numerous to count data for HPCs) are referred to as censored data. USEPA [9] provides general guidance for analyzing censored data, depending on the percentage of data points reported as less than or greater than a fixed value:

- If less than 15% of the data are affected, consider substituting values for the "nondetect" or TNTC data. For nondetect data, the detection limit or one-half of the detection limit may be appropriate values for substitution. For TNTC data, the threshold value could be substituted. If substitutions are made for more than 15% of the data points, USEPA recommends using nonparametric statistical tests for analysis.
- If 15% to 50% of the data points are affected, USEPA recommends applying special methods, such as the trimmed mean, Cohen's adjustment, and the Winsorized mean and standard deviation.
- If greater than 50% of the data are affected, USEPA recommends employing tests of proportions.

Tests for independence

Each of the statistical tests described in Table 5 assumes that data observations are independent of one another. This assumption is critical in maintaining the "power" of a statistical test. If data points are correlated, each data point does not bring independent information to the analysis, and the degrees of freedom for the test is effectively reduced. USEPA [9] recommends the Rank von Neumann Test for independence.

Step 5: Draw Conclusions From the Data

The fifth and final step of the DQA process involves performing the statistical hypothesis test(s) and interpreting the output as it relates to the DQOs. The decision will take the form of (1) rejecting the null hypothesis or (2) failing to reject the null hypothesis. This step should also include confirmation that the tolerable limits on decision errors were satisfied during the analysis. Additionally, USEPA [9] recommends evaluating the performance of the sampling design for future data collection efforts.

UTILITY CASE STUDY

Statistical methods were recently used to analyze data collected as part of a special-purpose monitoring program (SPMP) conducted by a large utility. The utility purchases water from a wholesale supplier and distributes the water to approximately 500,000 customers. The wholesale supplier uses a surface water source and provides conventional treatment. Chlorine is applied for primary disinfection. During the course of the one-year monitoring program, the wholesale supplier converted from free chlorine to chloramines for secondary disinfection. The utility's distribution system consists of pump stations, storage facilities, and more than 1,000 miles of distribution system piping. Most of the distribution system piping is unlined cast iron.

The utility's SPMP, which focused on microbiological parameters, was designed to achieve the following goals:

- Collect data to discern water quality changes resulting from changes in disinfection practices (i.e., the conversion from chlorine to chloramines for residual disinfection)
- Identify problem areas to aid in prioritizing maintenance or construction activities
- Provide a better understanding of biofilm susceptibility of the distribution system
- Provide baseline information about water quality in the general distribution system and in worst-case areas (e.g., low flow, dead ends, etc.) of the distribution system so that the effects of future changes in operational practices can be measured

Monitoring Program

The SPMP was designed to augment the sampling plan for regular TCR compliance monitoring. Specifically, the program focused on perceived worst-case areas of the distribution system. Sampling sites were identified with consideration of the following criteria:

- Areas in the vicinity of problematic TCR compliance sampling sites (i.e., TCR sites associated with samples that previously tested positive for coliform bacteria and exhibited high levels of heterotrophic bacteria or low chlorine residuals)
- Areas with relatively frequent customer complaints (i.e., taste and odor and/or muddy water) within the last five years that were not represented by a TCR sampling site
- Areas that typically had high THMs, reflecting high residence time, probable stagnation, and low flows
- Extremities of the distribution system near the closed valves at the boundaries of each pressure zone

In sum, 26 sites were selected for monitoring on a biweekly basis for a one-year period. The following parameters were measured during each sampling event:

- pH
- HPC
- Temperature
- Presence/absence of total coliforms
- Total chlorine residual
- Presence/absence of *E. coli*
- Free chlorine residual

Several additional parameters were designated *triggered parameters*, meaning they were only measured when certain conditions were met. Triggered parameters included nitrate, nitrite, ammonia, and total iron. During the first two quarters of sampling (February through August), nitrate, nitrite, and ammonia were analyzed if the chlorine residual measurement was less than 1.0 milligram per liter (mg/L). Total iron measurements were triggered by observation of red water, high bacterial counts (i.e., HPC>500), or a chlorine residual less than 1.0 mg/L. For the final two quarters of sampling (August through January), the free chlorine trigger level for nitrate, nitrite, ammonia, and total iron analyses was raised to 1.5 mg/L to trigger more frequent analyses of these parameters.

Statistical Analysis and Conclusions

The value of a statistical approach to distribution system water quality data is illustrated by the findings from the SPMP undertaken by the case study utility. Given the design and stated objectives of the SPMP, the value of the program is based upon its potential as a source of information beyond that available from routine compliance programs. The statistical analyses of the data from the yearlong SPMP were guided by three key questions regarding the overall value of the SPMP, the priority areas for remediation, and relationships among water quality parameters, each of which is discussed below.

Value of the SMP

Question: Is bacteriological water quality at SPMP sites significantly worse than water quality at TCR compliance monitoring sites?

The fundamental value of the SPMP rested on its ability to provide information beyond that available from routine compliance monitoring efforts. The SPMP aimed to identify worst-case areas of the distribution system for more intense scrutiny by the utility. Nonparameteric Wilcoxon Rank Sum Tests indicated that, as a group, SPMP sites had significantly higher HPCs than the collective TCR sites. This test verified that the SPMP monitoring provided valuable information about certain areas of the distribution system where bacteriological water quality tends to degrade.

Priority sites for remediation activities

Question: Among the SMP sites, is the bacteriological water quality significantly worse at some sites than at others?

HPC, though not a regulated parameter for this utility, serves as a valuable indicator of distribution system water quality. Nonparametric multiple comparisons were used to compare HPC data for pairs of individual sites. That is, the data collected from Site 1 were directly compared with Site 2, Site 3, . . . Site 26. Subsequently, Site 2 data were compared with data collected from Site 3, Site 4, . . . Site 26. Using this test, which compared the medians, variability, and number of data points for the two data sets, 4 of the 26 sites were identified as areas of concern. High levels of heterotrophic bacteria were consistently observed at these

4 sites; the multiple comparison indicated that HPCs at these sites were significantly higher than HPCs at the majority of other sites.

An occurrence of total coliforms (TC) in distribution system samples may indicate a bacteriological water quality problem. Samples from 6 sites tested positive for TC once during the course of SPMP monitoring. Using HPC from the yearlong monitoring program as a criterion, data indicated that bacteriological water quality was generally poor at 2 of these sites, marginal at 2 sites, and good at 2 sites. With one exception, all of these TC-positive samples were collected when free chlorine was the disinfectant residual.

Sites with high HPCs or occurrences of TC were identified as priority sites for further study and possible remedial activities (e.g., flushing, lining, etc.). In sum, 8 of the 26 sites in this "worst-case" sampling program were included among the priority sites. Additional investigation into water quality at these sites may involve further monitoring or examining whether maintenance activities (e.g., flushing, main repairs, etc.) coincided with occurrences of high HPCs or positive TC samples. Since chloramine disinfection was implemented only during the final quarter of sampling, additional data are needed to determine whether bacteriological water quality will improve as a result of chloramination. However, if significant improvements are not detected at these sites, the utility can devote resources to maintenance and construction activities to improve water quality. In this case, a statistical approach provides a reliable basis for operational decision making and guides the utility in making cost-effective decisions on the use of resources.

Relationships among water quality parameters

Question: How are the physical/chemical parameters (e.g., temperature, pH, disinfectant residual, and color) related to bacteriological water quality? What correlations exist among the physical/chemical parameters?

Statistical analyses of the data led to the following findings and conclusions:

- A Wilcoxon Rank Sum Test indicated that implementation of chloramines for secondary disinfection resulted in a measurable, statistically significant decline in HPCs in samples from both SPMP and TCR sites. When chloramines were used, total chlorine concentrations were also higher than when free chlorine was used for secondary disinfection. However, total chlorine concentrations did not correlate with bacteriological water quality, as measured by HPC. These findings support the idea that HPCs in this distribution system are more strongly influenced by the type of disinfectant than the total chlorine concentration.

- Since chloramine samples were limited to the November–January sampling quarter, the utility considered whether the decrease in HPC data during chloramination was attributable to temperature decreases rather than the change in secondary disinfectant. To better understand the influence of temperature on HPCs, TCR compliance monitoring data for the same period (i.e., November–January) in two different years were compared, but no correlation between temperature and HPCs was observed. This finding largely removed the influence of temperature and suggests that HPC reductions observed in the SPMP resulted from the use of chloramines rather than from declining water temperature.
- Significant correlations were not detected among other physical/chemical water quality parameters, specifically free chlorine, pH, color, and HPCs.
- Main size was an important parameter when the disinfectant residual was free chlorine. HPCs in samples from 8-in. and 10-in. mains were higher than in samples from 12-in. and 16-in. mains. No relationship between main size and HPCs was observed after the disinfectant residual was converted to chloramines.

SUMMARY

Using statistics, the utility confirmed that bacteriological water quality at the collective SPMP sites was significantly worse than the bacteriological water quality at the collective TCR sites; this result confirmed the usefulness of the program. Sites within the SPMP program with the poorest water quality were also identified and prioritized for maintenance (e.g., flushing, cleaning, etc.) and construction (e.g., lining, eliminating dead ends, etc.) activities. Additionally, the utility identified relationships among physical, chemical, and microbiological water quality parameters and confirmed that biofilms and microbially mediated corrosion are likely present in certain areas of the distribution system. Use of a statistical approach provided the utility with confidence in their findings, a basis for sound decision making, and the knowledge required to responsibly allocate resources.

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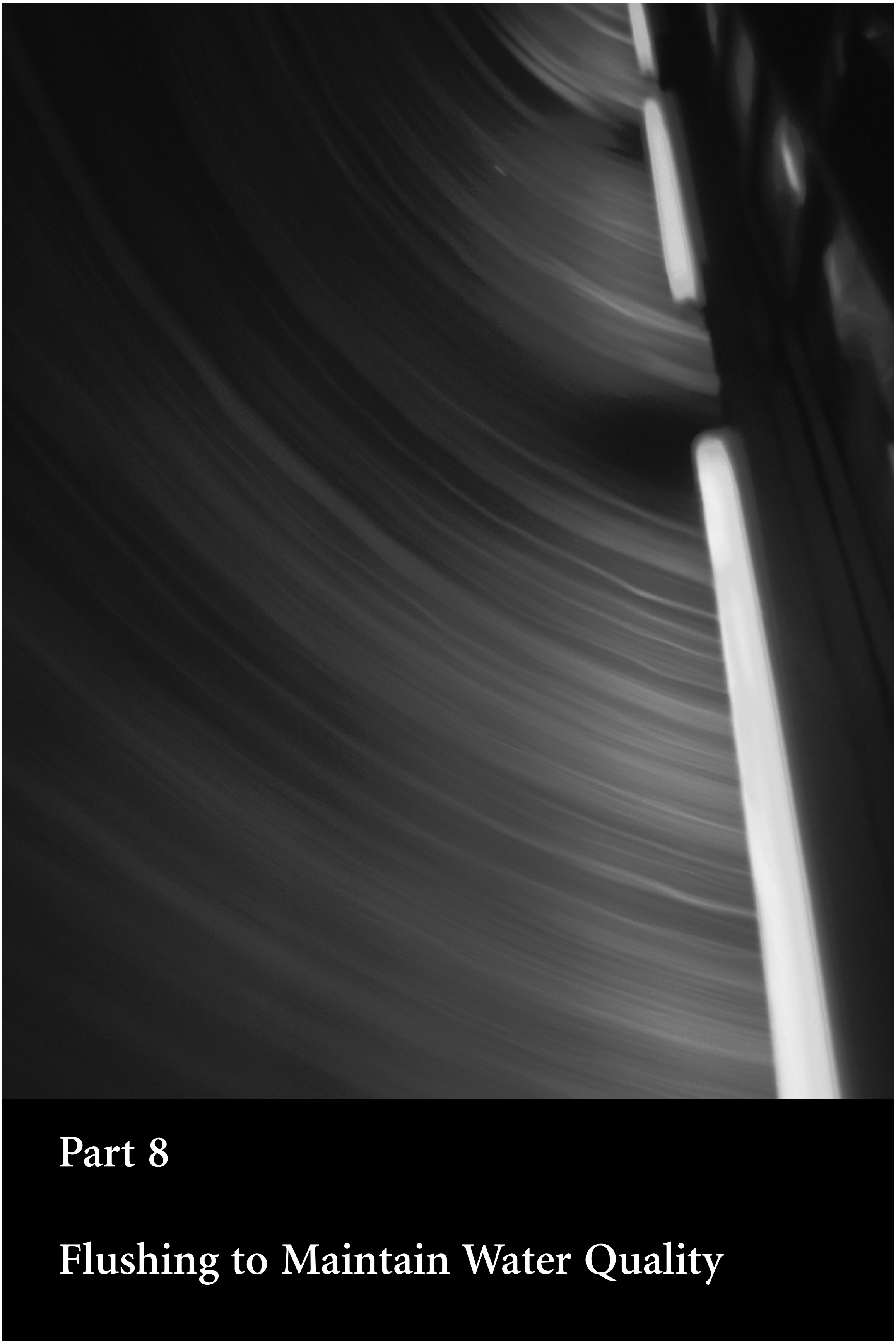

Part 8

Flushing to Maintain Water Quality

CHAPTER · 42

Developing and Implementing a Distribution System Flushing Program

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BACKGROUND

Utilities have long implemented flushing programs in one form or another and to varying extents within the distribution system. Generally, these programs have been established as corrective measures in response to customer complaints or to expel contaminants inadvertently introduced into the system.

WORKSHOP IDENTIFIES FOUR-STEP FLUSHING PROGRAM

Participants in the March 1998 AWWA Research Foundation (AwwaRF) study "Guidance Manual to Maintain Distribution System Water Quality" included utility representatives, consultants, members of the Project Advisory Committee, and members of the project team. Together, they devised a four-step flushing program that would meet the needs of utilities (Kirmeyer et al., 2000). The steps are as follows:

- Step 1—Determining the appropriateness of flushing as part of a utility maintenance program
- Step 2—Planning and managing a flushing program
- Step 3—Implementing a flushing program and data collection
- Step 4—Evaluating and revising a flushing program

Although the magnitude and frequency of flushing programs will vary from utility to utility, workshop participants agreed that flushing was one of many tools that should be considered best management practices for maintaining water quality in distribution systems. The group also concurred that flushing is an important part of a good distribution system maintenance program.

STEP 1: DETERMINING THE APPROPRIATENESS OF FLUSHING AS PART OF A UTILITY MAINTENANCE PROGRAM

Should a Utility Consider Routine Flushing?

The questions that follow can help utilities assess the appropriateness of a flushing system.

- Does the utility utilize an unfiltered surface water supply?
- Does the utility utilize an undisinfected groundwater supply?
- Does the utility use a water source with elevated iron and/or manganese?
- Does the utility experience positive coliform results or elevated levels of heterotrophic plate counts (HPCs)?
- Does the utility chloraminate?
- Is the utility planning to implement a treatment change that will alter the chemistry of the water?
- Does the utility experience frequent customer complaints related to water quality?
- Does the utility have difficulty maintaining a disinfectant residual in portions of the distribution system?
- Is the system lacking an aggressive valve/hydrant/tank exercising program?
- Would the utility consider the water entering the distribution system to be corrosive?
- Does sediment accumulate in storage reservoirs?

If any of these questions apply to a utility, it is probable that distribution system water quality improvements would be realized from implementing a routine flushing program. If the answer is "yes" to more than one of the assessment questions, not only should the utility consider implementing a routine flushing program, but it is probable that the system will benefit from more frequent flushing compared with a utility that answers “yes” to only one of the questions.

Systems that answer “no” to all of the questions should be able to document that the water quality issues listed are not occurring. It is not satisfactory just to assume that disinfectant residuals are maintained throughout the system on a year-round basis. Instead, monitoring should be conducted to document the microbial and chemical quality of the water, both seasonally and spatially. Suggested monitoring programs to evaluate the need for flushing and the effectiveness of existing flushing programs are discussed later in this chapter.

Assess the Feasibility of Flushing

After a utility has determined that distribution system water quality improvements could be realized from flushing, it is important to assess the feasibility of actually conducting an effective flushing program. The utility should consider the following questions prior to initiating flushing.

- Will hydraulic constraints prevent the achievement of desired flushing velocities?
- Is enough water available to flush at desired velocities for desired durations?
- What are the requirements for disposing of the water?
- What is the estimated cost (labor, power, equipment, and so forth) for conducting flushing?
- Is flushing the solution to distribution system water quality concern(s), or is it only part of the solution?
- What other operation/maintenance practices should be considered to address the specific water quality concern(s)?
- Does the entire system need to be flushed, or can water quality goals be achieved by focusing on certain portions of the system?

The answers should help the utility determine the degree of planning and level of effort that will be required to conduct an effective flushing program. If the level of effort is high and the potential water quality benefits are low (based on responses to questions in the previous section), then flushing may not be the best approach for maintaining water quality within a specific system. Other approaches, such as source water treatment, booster disinfection, pipe cleaning/lining, and increased tank fluctuations, may also address water quality concerns. A

Depending on a utility's water quality goals and treatment practices, measuring the disinfectant residual should be on the list of water quality parameters for a routine baseline monitoring program.

combination of approaches is often the most effective method of maintaining water quality throughout the distribution system.

STEP 2: PLANNING AND PROGRAM MANAGEMENT

Planning the flushing program is probably the most important step toward obtaining the desired water quality goals while minimizing unnecessary costs and undesirable secondary effects.

Determine Flushing Objectives

Flushing objectives may involve both water quality and hydraulic/maintenance considerations. It is imperative to decide what the water quality objectives are prior to flushing because the flushing approach used will likely vary depending on the specific water quality goal. Water quality concerns that can potentially be addressed through flushing include removal of accumulated silt/sediment from distribution system piping, reduction of chlorine demand throughout the distribution system, reduction of disinfection by-product (DBP) precursor materials, removal of accumulated biofilm, removal of contaminated water from a portion of the system, prevention of nitrification episodes, and reduction of customer complaints.

A flushing system can also address the following hydraulic/maintenance considerations: testing the structural integrity of the system under controlled (versus emergency) conditions, giving crews an opportunity to "operate" the system, providing opportunities to audit the system and associated appurtenances, and testing the hydraulic capacity of the system.

Additional Safety Considerations

- Ensure the flushing crews are properly equipped with personal protective equipment and the correct tools for flushing operations. These tools should be well maintained and replaced when necessary.
- Use the appropriate size crew for flushing operations. This typically involves a two-person crew. Individuals working alone are not efficient for flushing operations because of the distances between hydrants, valves, and so forth. While waiting for a main to clear, other crew members can be preparing the next section of main for flushing. They can also perform valve and hydrant maintenance work.
- Many of the valves that will be operated during flushing operations are located within paved portions of active streets. One crew member should control traffic while other crew members are operating distribution system valves.
- Traffic signs, cones, flags, and/or vehicles with warning lights should be used as necessary to divert traffic around flushing activities.
- Brightly colored safety vests should be worn by all crew members, especially when they perform the work at night.
- Keep the public away from ongoing flushing activities. Children are attracted to such activities and are vulnerable to injury.
- Use written procedures throughout the flushing operations to minimize unsafe situations and help coordinate the activities of the various crews involved.
- Open hydrant valves completely to prevent water discharging through the barrel drain at pressure, undermining the hydrant support.
- Open and close hydrants and valves slowly to prevent the development of dangerous pressure surges. A general rule of thumb is that for every 1 fps (0.3 m/sec) sudden decrease in flow velocity within a water main, a pressure rise of 50 to 60 psi (345 to 414 kPa) can be expected. Likewise, sudden increases in velocity can result in the development of low or negative pressure surges.
- Use well-restrained energy dissipators designed for that purpose to prevent damage to private and public property.
- Discharge flushing water directly to a sanitary or storm sewer whenever possible to avoid flooding of streets and underground electrical vaults. Where street flooding is unavoidable, use signs, flags, and other items to direct traffic appropriately.

Depending on the utility-specific water quality objectives and hydraulic/maintenance considerations, one or more flushing approaches may be appropriate. These are described here.

Three Flushing Techniques Are Available

There are basically three flushing methods: unidirectional, conventional, and continuous blowoff. Each approach has a specific use and can help to meet certain water quality goals. In addition, each approach can be implemented on a comprehensive, systemwide basis or on a narrower "spot" basis. This largely depends on the configuration of the system within the area of interest and the water quality goals for that particular area. The term "comprehensive" indicates that the flushing campaign addresses the entire system, whereas "spot" flushing is typically used to target an acute problem in an isolated area of the system or in an area where chronic water quality problems because of low demand and other factors have led to repeated customer complaints.

Spot flushing is typically implemented more frequently than comprehensive flushing. Because spot flushing focuses more on replacing the bulk water (versus cleaning the pipe), water quality benefits typically last only a short time. However, in the absence of any flushing program, it may still be beneficial to conduct spot flushing at dead ends. Flushing an area with low demand on a monthly or quarterly basis, for instance, can be considered spot flushing. A brief discussion of flushing approaches, advantages, and disadvantages is provided here.

Unidirectional flushing (UDF)

According to Antoun et al. (1997), UDF consists of isolating a particular pipe section or loop, typically through closing appropriate valves and creating a single-direction flow. It is carefully engineered with consideration of the size of the flushing crew, duration of flushing, equipment availability, and location of water sources such as water treatment plant, storage tanks, and booster stations. UDF can be implemented as a comprehensive, systemwide flushing effort to prevent water quality deterioration or on a spot basis in response to a specific water quality concern.

The term "unidirectional" is often associated with a velocity of ≥5 fps (1.5 m/sec) (Oberoi, 1994), which is thought to be adequate for the removal of biofilm and corrosion products and other debris attached to the pipe walls. However, the concept of isolating pipe segments and flushing in a sequential manner from the source to the periphery (i.e.,

Distribution system maps can be used to identify flushing loops, valves to be opened or closed, locations of pump stations and hydrants, pressure-reducing valves, and other facilities.

unidirectional) can be practiced at lower velocities to achieve different water quality goals, such as removing loose sediments, restoring chlorine residual, and reducing tastes and odors.

Utilities must take the initiative to determine the most appropriate velocities for meeting their individual water quality goals. An AwwaRF study (number 2606) titled "Establishing Site-specific Flushing Velocities" is currently under way both to characterize accumulated materials and to determine required velocities for lifting, scouring, and removing contaminants from the distribution system.

UDF can lead to both water quality and hydraulic improvements. Benefits associated with a systemwide UDF program may include the following:

- Reducing the management hours required to oversee the program implementation, because all the needed information is presented on the flushing maps and accompanying step-by-step flushing instructions
- Allowing for simultaneous implementation of preventive maintenance activities
- Using less water than conventional flushing (savings of greater than 40% by some estimates) (Oberoi et al., 1997)
- Standardizing procedures, which provides a uniform basis of comparison with future flushing events
- Reducing troubleshooting efforts because of searches for closed valves that are supposed to be open
- Allowing chloraminated systems to quickly and effectively implement flushing during conversion to free chlorine or to prevent the onset of nitrification.

Conventional flushing

Conventional flushing is usually implemented with little, if any, preplanning. Conventional flushing consists of opening hydrants in a specific area of the distribution system until preselected water quality criteria are met. These criteria could include detectable disinfectant residual, reduction/elimination of color, reduction in turbidity, and so forth. It is important to note that valve isolation is not part of conventional flushing. Consequently, flushing velocities are not maximized, because water to the hydrant often flows from several mains, with the resultant velocity in each individual main remaining low. In contrast, by using valve isolation for UDF, water is forced through a single main at higher velocities. Further, because the water used to flush a particular main during conventional flushing may not originate from a segment that has already been flushed, the cleaning efficiency is not maximized, and contaminants can be transported from one main to another. The primary water quality improvements that can be achieved through conventional flushing may include restoration of disinfectant residual and expulsion of some of the poor-quality water in specified areas of the system.

If these are the primary water quality goals within a specific portion of the system, conventional flushing may be adequate. Conventional flushing typically requires less planning than UDF; however, the opportunity to combine flushing with valve inspection/exercising programs is diminished. Additionally, conventional flushing has resulted in the following water quality problems: increased customer complaints during and immediately after implementation, a considerable waste of water, short-lived water quality benefits, and the potential for increased coliform occurrences following flushing.

Continuous blowoff

For utilities that have numerous dead ends and severe water circulation problems, continuous blowoff, or bleeding of water, may be conducted to force a low-velocity flow through a small portion of the system. Typically, because of hydraulic restrictions associated with the use of blowoff valves on dead-end lines, velocities of less than 1 fps (0.3 m/sec) are attained. Therefore, unless used in conjunction with hydrants, adequate velocities may not be achieved to remove accumulated sediments or to provide scouring. The use of continuous blowoffs can help utilities restore disinfectant residuals and reduce water age. Obviously, this practice can result in the use of large quantities of water, without providing a solution to the cause of the water quality problem. Although continuous blowoff may be necessary to reduce detention time, more permanent solutions should be considered, such as looping dead ends or installing smaller-diameter pipes in lieu of larger pipes to meet future demands.

Preliminary Program Development Will Vary

Depending on the type of flushing program to be carried out, the degree of planning and program development will vary. This section focuses on developing a UDF program, because it is widely accepted that a UDF program will produce the greatest long-term water quality improvements. Additionally, the UDF approach likely requires the greatest level of planning. The information provided here can be scaled down to assist utilities with developing the other types of flushing programs as well.

Obtain maps and review hydraulic models

The first step in developing a UDF program is to gain a thorough understanding of the distribution system's hydraulics and flow patterns. If a calibrated hydraulic model is not available, it will be necessary to obtain and review distribution system maps along with sewer and stormwater maps. The distribution system maps will be used to identify flushing loops, valves to be opened or closed, locations of pump stations and hydrants, pressure-reducing valves, plus other distribution system facilities. The sewer and stormwater maps will be needed to identify the locations to which the flushed water will be discharged. At this stage, it should be verified that adequate water and pressures are available to attain desired flushing velocities to meet water quality goals.

Develop a list of stakeholders

Once flushing loops have been identified, it will be very beneficial to develop a list of stakeholders that may be affected by flushing activities or that can facilitate the successful implementation of a flushing program. Such stakeholders may include preventive maintenance program directors, fire department(s), sewer/stormwater departments, customer service/public education departments, adjacent/interconnected water systems, and the water quality department/laboratory.

The flushing program should be reviewed with potential stakeholders to avoid problems that can be associated with crossing interagency "turf" boundaries and to obtain concurrence regarding water disposal issues, hydrant use, and so forth. Additionally, any requirements associated with treating the flushed water before discharging it to storm sewers or fish-bearing streams should be clearly identified. Potential parameters of concern with regard to disposal issues are discussed in the section titled "Collect Data to Assess Program Success." Additionally, the *Guidance Manual for Disposal of Chlorinated Water* (Tikkanen et al., 2001) addresses the disposal of chlorinated water in compliance with federal, state, and provincial regulations while minimizing the impact of these requirements on operations.

Combine flushing program with other preventive maintenance programs

A flushing program should not be a stand-alone effort that requires additional crews for a single purpose. Rather, implementation of a flushing program should be coordinated with other distribution system preventive maintenance programs. In this way, labor savings can be realized, and a more comprehensive view of actual hydraulic conditions of the system can be attained. Combining maintenance programs can help the utility meet numerous water quality, operational, and maintenance goals. Additional maintenance programs that could be carried out simultaneously include tank cleaning; valve/hydrant exercising, survey, and inventory; fire department hydrant testing; sewer flushing; and street sweeping/inlet cleaning.

Notify the public

Citizen sensitivity to apparent waste by a public agency, the sight of a water utility crew letting hydrants run into the street, and the potential for discolored water during the flushing event can trigger numerous complaint calls. Therefore, public notification is essential to implementing and maintaining an effective flushing program. It is important for the public to understand the reasons why flushing is conducted, such as improving water quality, decreasing the reliance on chemical treatment and chemical use within the distribution system, improving system hydraulics, ensuring that water can and will be made available in emergency situations, and so on. Most people can appreciate these objectives and will be supportive of the utility's efforts.

A good public notification program will educate and inform not only customers but also other utility staff (especially field crews who will have direct contact with the public), regulators, and other stakeholders. Adequate notice should be given whenever possible using a variety of media to reach as many parties as possible. Depending on the size of your utility, one or more of the following notification methods may be appropriate: mailer/bill inserts, newspaper notices, television and/or radio announcements, door hangers, telephone calls, electronic postings, and posting signage in the neighborhood during flushing.

All utilities should identify sensitive users, such as hospitals, high-tech industries with on-site water treatment requirements, and kidney dialysis patients, and try to accommodate their needs. In these instances, adequate notice of flushing is imperative. Perhaps flushing could be conducted at night to minimize effects on sensitive users, or perhaps the utility could alter the point of service to the customer.

Utilities should also give customers an opportunity to provide feedback regarding the immediate effects of flushing on water quality at the tap, problems that may be experienced, and potential observations related to water quality improvements since flushing has been implemented. This type of information can be used by the utility to better plan its programs and to evaluate the effects and benefits of flushing.

STEP 3: PROGRAM IMPLEMENTATION AND DATA COLLECTION

Once a utility has identified water quality and hydraulic objectives, potential constraints to flushing, sensitive users, several methods for notifying the public and other stakeholders, and other preventive maintenance programs that can be combined with a flushing program, the actual flushing program can be implemented.

Developing and Implementing a UDF Program Involves Dividing System Into Loops

The first step is to divide the distribution system into individual loops. These are sections within the distribution system, starting at the water source(s) and ending at the system's periphery, to be flushed in sequence. Each loop consists of a manageable section, the size of which is determined with consideration of flushing crew size, duration of flushing, equipment availability, and location of water sources such as water treatment plant, storage tanks, and booster stations. The goal is to complete flushing each individual loop or multiple loops during the crew's predetermined work shift. This will allow for reopening all valves used for isolation while flushing the particular loop, which avoids keeping normally open valves closed for extended periods. An experienced two-person crew can often flush 1 mi (1.6 km) of pipe per day using a UDF approach.

Refining and Evaluating a Flushing Program

Utilities may want to consider the following questions to evaluate and refine their own flushing programs.

Were Water Quality Objectives Met?

- Document improvements in water quality (e.g., improved chlorine residual, decreased turbidity, fewer customer complaints, less time to reach water quality objectives from year to year).
- If objectives were not met, was the situation improved or worsened?

What Were the Estimated Costs/Savings Associated With the Program?

- Assign actual costs when possible (e.g., labor, power, equipment, disposal requirements, planning/notification efforts).
- Estimate savings from conducting maintenance activities simultaneously.

Were There Positive Secondary Effects Because of the Flushing Program?

- Were operating costs reduced (e.g., lower chemical dosage requirements, reduced power costs from decreasing pipe friction, improved valve/meter/hydrant life)?
- Was customer perception improved?
- Is there potential for increased industrial use as a result of water quality improvements?

Were There Negative Secondary Effects From the Flushing Program?

These might include stirred-up portions of the system in uncontrolled fashion, increased chlorine residual that resulted in customer complaints, release of bacteria into the water column, exposed new surface of tubercles, and negative public perception or waste of water.

Following loop delineation, desired flushing velocities must be determined. For practical reasons, large transmission mains with diameters greater than 24 in. (600 mm) are generally not flushed. Under the strictest definition of UDF, flushing velocities throughout the system must be ≥5 fps (1.5 m/sec) (Oberoi, 1994). However, a unidirectional approach can be used at different velocities, depending on the desired water quality goals and system constraints, which may limit attainable velocities.

The next task in developing a UDF program is to prepare step-by-step flushing procedures, which provide precise instructions with regard to the sequence of valve and hydrant opening and closing. For each loop, an average of 10 to 12 steps can be developed. Depending on the complexity of the system, each step can be complemented with an individual map that clearly illustrates, in color, the valve and hydrant status (open or closed) during each step. Individual maps for each loop can be laminated and used by the flushing crew. Maps may be developed from geographic information system (GIS) coverages, from computer-assisted design files, or even by scanning hard copies of maps that are not available electronically.

Prior to program implementation, it is recommended that a "worst-case" loop scenario be tested to assess the extent of repairs, if any, that may be required during program implementation. Ideally, the repair crew should conduct preventive maintenance activities for those valves and hydrants to be exercised during UDF implementation.

The final step is program implementation. This should be carried out with consideration of public and flushing crew safety. An effective UDF program requires not only good design but also proper execution. Some of the key techniques for an effective program are as follows:

- Flushing should progress from the water treatment plant or well to the system's periphery.
- Flushing should progress from larger to smaller mains.
- As previously indicated, flushing velocities should ideally be at least 5 fps (1.5 m/sec) if pipe scouring is desired.
- Valves and hydrants should be exercised prior to flushing (together with a vigorous maintenance program) to minimize interruptions during flushing.
- Crews should be properly trained and equipped.
- Flushing should ideally be performed during late-night and early-morning hours (11 p.m. to 5 a.m.) to minimize effects on customers and take advantage of high pressures.

Take Safety Into Consideration

Safety is of paramount importance during flushing operations. Poor safety practices can result in damage to property and injury to both crew members and the general public. This is especially true with flushing velocities ≥5 fps (1.5 m/sec). As with any field operation, there are a number of general safety considerations to be incorporated into flushing program activities.

Foremost is an active safety program to ensure employees are knowledgeable about typical water utility procedures and associated hazards. This program should include a system for rewarding employees for good safety records and encouraging suggestions for improving on-the-job safety. Additional safety considerations are provided in the sidebar on page 869.

Collect Data to Assess Program Success

A data collection program enables the utility to determine whether flushing objectives are being met and to assess secondary effects that may have occurred as a result of flushing. The program assumes that flushing is being carried out as an active maintenance practice to improve distribution system water quality. Data collection has been divided into three categories: baseline, during flushing, and postflushing.

Baseline

“How do I document the effectiveness of a flushing program?” was one of the most common questions posed at the flushing workshop. The answer may well be in monitoring. It is recommended that utilities conduct rigorous baseline monitoring throughout the distribution system—from source to periphery—including reservoirs, sample stands, inlets, and dead ends, to assess water quality conditions prior to implementing flushing. Collection of baseline data will help the utility to identify problem areas of the system that can most benefit from flushing, potentially avoiding the expense associated with flushing all portions of the distribution system. Parameters that are of concern to the utility should be monitored on a seasonal basis. Historical data can also be used to help set a baseline condition. Depending on the individual utility’s water quality concerns and treatment practices, the following list of water quality parameters should be considered for a routine baseline monitoring program: disinfectant residual; color; pH/alkalinity; coliform; iron/manganese; DBPs; disinfectant/corrosion control dosages at the treatment plant; water quality–related customer complaints; turbidity; temperature; inhibitor concentration; HPC (using R2A media); conductivity; and ammonia, nitrate, and nitrite (for chloraminating utilities).

Several of these parameters can be analyzed on-site with only minimal cost to the utility. Monitoring frequency will vary based on available budget, the size of the system, and water quality variability throughout the system. Monitoring should be conducted on a seasonal basis at a minimum. Once problem areas are identified, flushing loops can be developed, and velocities can be selected to produce the desired water quality results.

It is recommended that utilities avoid implementing more than one system change at a time. For example, it may be difficult to assess the effects of a new flushing program on distribution system water quality if a treatment change—such as switching disinfectants—is made simultaneously. Changes in bulk water chemistry can affect scales that have accumulated on pipes, and it may be difficult to separate out the effect of flushing at a certain velocity to achieve scouring versus the effect of softened scales because of changes in water chemistry.

Data related to costs, including labor, equipment, water use, and public notification, should also be diligently tracked so that a cost–benefit assessment can be conducted and the flushing program can be refined as needed. The AwwaRF research report “Cost and Benefit Analysis of Flushing” can help utilities assess individual flushing programs.

During flushing

Water quality data collected during flushing should focus on the specific water quality objective (e.g., disinfectant residual, pH, turbidity). Parameters should be measured in the field. The data should be collected at the beginning, middle, and end of a flush. Data collected from the beginning of a flush should be taken after the hydrant barrel has been cleared. The time to reach the desired water quality goal (or time to clear) should be recorded, and this can be used as the flush endpoint. Ideally, at least one pipe volume will be discharged from the system prior to ending the flush. This will allow all suspended material to be removed from the main. Pressure and flow data should also be collected during a flush, and pressures should be checked upstream of the flush to ensure that pressures of at least 20 psi (138 kPa) are being maintained. In addition to pressures and flows, valve/hydrant locations and conditions should be recorded to update distribution system maps.

Additionally, data should be collected so that chlorinated water can be disposed of properly. Depending on local or regional requirements, dechlorination or pH adjustment may be necessary, and there may be limits to solids content and dissolved oxygen levels that can be discharged to fish-bearing streams. Appropriate data should be collected so that treatment of the discharged water can be accomplished effectively. Data related to downstream flows into storm or sanitary sewers should also be collected.

After flushing

Postflushing data collection has been divided into short-term and long-term monitoring. Short-term monitoring is necessary to assess secondary effects (positive and negative) that can be attributed to the flushing. It is recommended that the following parameters be measured upstream of the flushed area and from within the flushed vicinity: HPC (R2A media), turbidity, disinfectant residual, coliform, corrosion products, color, and other parameters of specific interest.

It is possible that flushing will have "stirred things up," although practicing UDF rather than conventional flushing and discharging at least one complete pipe volume should minimize this problem. However, it is incumbent on the utility to ensure that flushing activities have not inadvertently exposed customers to temporarily high levels of HPCs, coliform, and so forth.

Long-term postflushing is the same as baseline monitoring and should be continued at the same locations so that the benefits/drawbacks of flushing can be assessed.

Take the Time to Manage Data

Each utility must take the time to record and evaluate the data collected before, during, and after flushing. Only by evaluating the data collected will the utility be able to justify the expense associated with a routine flushing program. Data analysis will also enable the utility to refine the flushing program, potentially reducing costs and water use, while still achieving the desired water quality improvements. Flushing data that should be routinely analyzed relate to water quality, flows and pressures, maintenance issues, customer complaints, and costs.

Comparison of short-term and long-term (baseline) water quality data from year to year will help to determine whether flushing has led to improved water quality conditions on a systemwide basis or just within specific problem areas. The time needed to reach water quality objectives (or time to clear) during the flush should be compared from year to year to determine whether required flush times have decreased. This type of analysis is especially important for utilities that are flushing only dead ends.

If time to clear is not decreasing over time, it is likely that no long-term benefits are being realized from the flushing program. Additional operational, maintenance, or source water treatment approaches may be necessary to improve water quality conditions. The type, number, and location of customer complaints should be reviewed as well, to determine whether flushing has improved certain water quality conditions at the tap or, conversely, has created customer dissatisfaction. Additional information related to evaluating/refining a flushing program is provided under step 4.

Some flushing crews carry laptop computers in the field, and data can be instantly entered for later evaluation by the appropriate personnel. Data related to hydrant/valve location, position, and condition can be entered into the system's GIS or hydraulic models to refine the calibration process.

STEP 4: EVALUATE PROGRAM AND REFINE IT AS NEEDED

Utilities should be able to justify the need for their flushing program (or, conversely, to justify the lack of a flushing program). It is possible that not all anticipated benefits will be realized instantly, and experimentation with velocities, duration, and frequency may be required to develop a program that is within budgetary constraints yet produces documentable improvements in water quality. For many utilities, maintaining a disinfectant residual throughout the system is justification enough for conducting routine flushing. For some utilities, knowing that they are doing everything possible to prevent water quality degradation and to maintain water quality conditions as close as possible to those at the point of entry is of great enough benefit to their customers that the flushing program is worth the time and expense. To conduct such an evaluation, it will be necessary to carefully document the costs, potential savings, benefits, and problems, as well as the secondary effects that can result from a flushing program. See the sidebar on page 876 for more information on evaluating and refining a flushing program.

It is up to the utility to assess the advantages and disadvantages associated with its individual flushing program and to make the necessary changes in program procedures so that negative secondary effects can be minimized. For example, public education/notification efforts may need to be increased if the public complains about wasted water. A UDF approach may be required if conventional flushing results in uncontrolled "stirred-up" water in other portions of the system. Flushing times may not be adequate if scoured bacteria are not being discharged from the system. Velocities may be too high if tubercles are being exposed, causing red water problems.

FLUSHING PROGRAM BENEFICIAL TO MOST UTILITIES

The workshop participants agreed that a routine flushing program is part of a good overall maintenance program. However, utilities should not rely on flushing as a cure for chronic problems. Adequate source water treatment may be necessary to permanently prevent certain types of water quality deterioration.

Also, not all utilities will require flushing. However, before flushing is excluded from a maintenance program, these utilities should have an aggressive water quality monitoring program (well beyond that required by regulations) that documents that water quality conditions are optimized throughout the system.

If conducted in conjunction with other preventive maintenance practices, flushing may not require drastic increases in operating/ maintenance budgets. In fact, savings may actually be realized.

The flushing approach, velocity, frequency, and duration will be specific to individual water quality concerns, hydraulic conditions, source water treatment, and system design. Utilities should select a flushing velocity based on specific water quality objectives. Experimentation with velocities, duration, and frequency should be part of a flushing refinement program.

Planning, public notification, and communication with stakeholders should be a mandatory part of all flushing programs. Many of the negative secondary effects discussed in this chapter can be avoided through proper notification of potentially affected agencies and customers.

Data collection before, during, and after flushing is imperative to understanding the benefits, costs, and secondary effects that occur because of flushing. If adequate data are not collected, program refinement and a cost–benefit analysis cannot be conducted.

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CHAPTER · 43

Managing Water Quality Through Uniform Directional Flushing

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BACKGROUND

CDM has been assisting the Public Works Commission of the city of Fayetteville, N.C. (PWC), in the planning and design of its water distribution infrastructure for more than 10 years. Figure 1 provides a layout of the PWC water distribution service area. Over the past few years, PWC has received complaints of discolored water from the Vanstory Hills, Montclair, and Haymont areas during the warmer months (generally starting Memorial Day weekend). In early May 2001, PWC received an extremely high number of complaints in these areas, leading it to authorize CDM to conduct a discolored water evaluation to identify the potential causes of the discolored water events. The evaluation identified three contributing factors:

- *Deterioration of unlined cast-iron pipe, a major source of color-causing material.* Customers in the subject area generally described the discolored water as red or brown, colors often associated with corrosion and tuberculation of aged, unlined cast-iron pipe. CDM reviewed data provided by PWC on the construction material and installation date of the water mains in the subject area as well as in the general area between the water treatment facilities (WTFs) and the distribution system repump facility. The evaluation showed that a significant portion of unlined cast-iron pipe exists in the areas of complaints and in the transmission lines that convey water into the subject area.
- *Incomplete flushing operations.* PWC had previously been flushing its water distribution system. However, CDM's evaluation of the flushing program revealed some areas for improvement in PWC's historical flushing practices. These improvements included

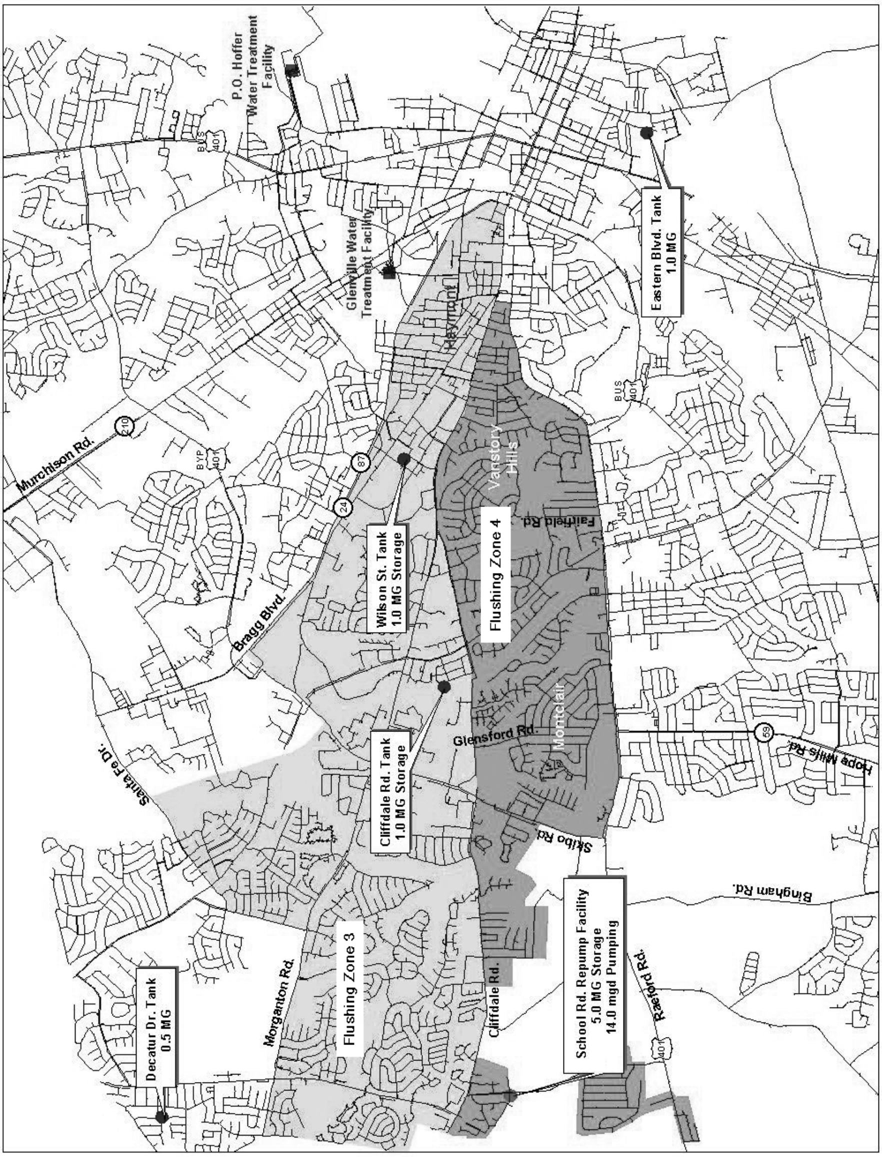

Figure 1 Distribution system and flushing zones

flushing the entire system out to the farthest extremities, designing a uniform directional flushing (UDF) program, eliminating flow reversals, achieving proper flushing velocity, accounting for the various pressure sources during flushing activities, and improving record keeping of flushing activities.

- *Pump operational impacts*. CDM evaluated the operation of PWC's various pumping facilities, including the high-service pumps. The evaluation showed that during the first high-water-demand periods of the year, the production from the School Road repump facility was rapidly increased. This sudden increase may have resulted in transients, surges, and flow reversals, leading to subsequent resuspension of materials in the system, thus reintroducing discoloration-causing materials into the complaint areas.

In order to address these causative factors and to assist PWC in its mission to provide the highest level of water quality and customer service, and to prudently manage its investment in water distribution infrastructure, CDM recommended that the following three tasks be performed:

- Task A: Refine the existing flushing program. It was recommended that PWC learn from the discolored water issues of past years, and not discontinue flushing before reaching the distribution system extremity. Additionally, it was recommended that PWC redefine and resequence its flushing zones to facilitate a more thorough system flushing, using UDF.
- Task B: Develop a predictive maintenance program for scheduling replacement and maintenance of water distribution pipe.
- Task C: Develop pumping sequence strategies at the School Road repump facility that provide gradual flow changes and reduce transients/surges in the western portion of the distribution system.

These tasks evolved into a distribution maintenance development project involving two phases. The primary objectives of Phase 1 were to minimize future complaints of discolored water in the Vanstory Hills, Montclair, and Haymont areas by improving the flushing program and determining the most effective method to operate the School Road repump facility and WTF high-service pumps in order to minimize the potential for discolored water. This chapter describes the enhanced UDF program and its results. Phase 2 will expand refinement of the flushing program to all areas of the distribution system and will include development of a predictive maintenance program.

BENEFITS OF AN ENHANCED UNIFORM DIRECTIONAL FLUSHING PROGRAM

For many years, potable water treatment purveyors have been aware of the numerous benefits of flushing out water distribution system piping networks. However, some negative issues that may develop when using conventional flushing programs have caused many water purveyors to minimize and/or abandon their flushing activities. These negative issues include:

- Zero or only marginal improvement in water quality
- Ineffective use of manpower resources, with minimal positive impacts
- An increase of complaints from customers due to stirring up the system

A UDF program is a proactive, systematic, engineered approach to flushing that addresses all of these concerns and decreases the municipality's need for emergency or reactive flushing. As the U.S. Environmental Protection Agency (USEPA) and the general public focus more attention on water quality in the distribution system, an effective flushing program will become increasingly important.

There are many driving factors and benefits for implementing a UDF program. Typically, a water purveyor will implement a UDF program in order to address one or two of these issues. However, the program can also deter other issues from arising. The following list summarizes many of the reasons for implementing a UDF program and the long-term benefits that an effective UDF program will provide.

- Expel stagnant water from dead-end lines.
- Increase chlorine residual within the distribution system by expelling older water and by removing elements within the distribution piping that consume chlorine.
- Decrease the amount, and subsequently the costs, of disinfectant chemicals used at the WTF by removing elements within the distribution piping that consume chlorine.
- Expel contamination from the distribution system.

- Respond to customer complaints regarding discolored water and/or taste and odor.
- Increase the carrying capacity of tuberculated/encrusted distribution system piping.
- Fulfill the requirement for chlorine/chloramine conversion cycles.
- Decrease the potential for nitrification occurrences in the distribution system.
- Decrease positive coliforms or elevated HPCs.
- Remove sediment accumulation in large pipes or storage tanks.
- Reassure USEPA and the general public of attention to water quality in distribution systems.

Uniform directional flushing of the distribution system can improve a water utility's water quality in several ways. It removes biofilm and other bacteriological growth that may cause positive coliforms as well as taste and odor concerns. UDF also removes sediment, corrosion products, and encrustation that may cause turbidity or discoloration. It improves water quality by introducing fresh water with higher chlorine residual into the water main and improves system reliability by preventing tuberculation and maintaining adequate flow. Through its implementation, it can also assist a utility with collecting data on valves, blowoffs, and hydrants, and with improving the accuracy of data for distribution system maps and the GIS database.

GENERAL UDF GUIDELINES

A highly effective UDF program involves a reasonable amount of up-front planning, sequencing, and design. Water utilities find that these efforts at the planning stage provide significant benefits in the long term; a well-designed UDF program can continue to be utilized for many years. The following general guidelines are often used to design an effective UDF program:

- Close valves for isolation of a line and area.
  - Do not open a line that will allow material to go back into the inner areas that have been flushed.
  - Do not open a line that will allow unflushed areas to be drawn from during flushing. This will pull in unclean water and may "shake up" the unflushed area.

- Flush outward from all supply and pressure sources, such as pumps, elevated tanks, and WTFs.
- Isolate individual supply and pressure sources (such as storage tanks) as much as possible.
- Flush (as often as possible) a line from a line that has already been flushed.
- Flush the system outwardly to the extremities in consecutive order.
- Flush in consecutive order and finish an area before beginning another area. Flushing effectiveness is lost if crews skip a section and come back later.
- Use the same “clear water” test procedure for consistency, such as a white Styrofoam cup or a clear plastic cup for a visual check of water color.
- Maintain system pressures in other portions of the distribution system (greater than 20 pounds per square inch) during flushing.
- Establish a standard protocol for recording flushing activities in GIS and MIS databases. Additionally, a standard protocol should be developed for reporting and addressing all deficient hydrants and valves or other issues noted during flushing activities.

UDF GUIDELINES SPECIFIC TO PWC

In order to implement a UDF program specific to PWC’s distribution system, several guidelines specific to PWC were developed. These guidelines included an identification system, consideration of distribution system demand distribution patterns and the influence of sources of flow, modeling of flushing scenarios, standards for field activities, and considerations during loop design.

Identification System

In order to develop the UDF design for the various flushing loops, each element of the distribution infrastructure required a specific identification number (ID). The ID system was needed to provide a unique letter identifier for each element of the distribution system. It is based on the actual component (valve, hydrant, or pipe), followed by the randomly generated ID number established via PWC's maintenance management system, which is the Cityworks system developed by Azteca. Using the ID system, specific loops and directions for flushing each loop were established. The ID system developed for the UDF program can be used for other PWC projects as well.

Distribution System Demand Distribution and Flow Patterns

To most effectively implement a UDF program, it is important to understand the impact of the various sources of flow throughout the water distribution system. Flow reversals can result in water from an unflushed portion of the distribution system entering into the portion of the system being flushed, or into a portion that has already been flushed. This can significantly decrease the success of the UDF program. To help ensure that flow reversals do not occur during flushing activities, demand distribution and flow patterns must be verified.

General system description

As shown in Figure 1, the PWC water system is divided into two primary service areas: the high service area (hydraulic grade line = 370 feet) and the low service area (hydraulic grade line = 240 feet). The discolored water subject area is located in the high service area where flow is generally in an east-to-west direction from the WTFs to the western part of the system. Both the Glenville WTF and the Hoffer WTF feed the high service area. Distribution system storage is provided by the Wilson Street elevated storage tank, Cliffdale Road elevated storage tank, and School Road ground storage/repump facility. The Wilson Street and Cliffdale Road elevated storage tanks each have a capacity of 1 million gallons. The School Road ground storage/repump facility consists of a 5-million-gallon ground storage tank and 14-million-gallons-per-day (mgd) pump station.

Approach

Using the hydraulic model developed by CDM in February 2002 as part of the PWC water system master plan update, CDM evaluated the flow patterns under various operating scenarios. The hydraulic models simulated the PWC distribution system with various production rates at the Hoffer and Glenville WTFs, and various pumping rates from the School Road repump facility. Water demands were based on current water demands of the existing system. A summary of the various model simulations conducted is presented in Table 1.

In general, under average demand and regular operating conditions, Skibo Road is the break line for the influence of the School Road repump facility. As the School Road repump facility flow is increased without production from the WTFs being increased, its influence on flow patterns east of Skibo Road along Raeford Road increases. If not appropriately accounted for during UDF loop development, a flow reversal could occur.

Modeling of Flushing Scenarios

In order to verify availability of water from assumed sources and to minimize the potential for improper flow directional patterns during flushing activities, CDM modeled PWC's water distribution system under various flushing conditions.

The hydraulic modeling indicated that PWC will not be able to maintain adequate pressures throughout the distribution system if attempting to flush areas west of Skibo Road without using the School Road repump facility at demands higher than maximum day demand (MDD). A modeling scenario was developed to verify that a change of flow direction would not occur when flushing crews performed UDF activities in the Vanstory Hills subdivision. In order to simulate this

Table 1 Demand distribution patterns model runs

Scenario	Demand	Glenville (*mgd*)	Hoffer (*mgd*)	School Road (*mgd*)
1	ADD	7.7	14.2	2.0
2	ADD	5.5	14.2	5.0
3	MDD	10.1	21.9	8.0
4	MDD	6.6	23.5	10.0
5	MDD	7.6	18.3	14.0
6	PHD*	7.6	18.3	14.0

*Demands in excess of MDD are supplied via elevated storage.

ADD = Average Day Demand, MDD = Maximum Day Demand, PHD = Peak Hour Demand condition, CDM placed a 530-gallon-per-minute (gpm) demand along one of the 6-in.-diameter mains in the subdivision, under an MDD condition. This 530-gpm demand represents the flow required to achieve a scouring velocity of 6 feet per second (ft/sec) in the 6-in.-diameter main. The Glenville WTF was producing 13.7 mgd, the Hoffer WTF was producing 25.4 mgd, while the School Road repump facility was not operating. Flow patterns did not change under this scenario. The model indicated that water would be conveyed from the east on Cliffdale Road from the WTFs. Water also was shown to come from the north along Northview/Fairfield Drive. No water was shown to circle around the system and enter from the south or west side of the service area, which would have introduced unflushed water into the flushing area.

Various modeling scenarios were also developed to evaluate the flushing needs along Raeford Road, west of Skibo Road, with the School Road repump facility out of service. The first condition modeled was with PWC at one-half of the average day demand (ADD), with Glenville WTF producing 6.4 mgd and Hoffer WTF producing 16.7 mgd. CDM introduced a flushing demand of 8,400 gpm into the model at the intersection of Raeford Road and Reilly Road. This 8,400-gpm demand represents the flow required to achieve a scouring velocity of 6 ft/sec in the 24-in.-diameter main. Under this scenario, the model indicated that pressures remain adequate (above 40 psi) throughout the distribution system, and that flow from unflushed pipes into the flushing area does not occur.

The next condition modeled was with PWC at the ADD, with Glenville WTF producing 8.5 mgd and Hoffer WTF producing 27.2 mgd. CDM introduced a flushing demand of 8,400 gpm at the intersection of Raeford Road and Reilly Road. Under this scenario, the model indicated that pressures remain reasonably adequate throughout the distribution system, and that flow from unflushed pipes into the flushing area does not occur.

Additionally, CDM modeled a condition with the system at the MDD, with Glenville WTF producing 14.2 mgd and Hoffer WTF producing 32.0 mgd (which is its maximum rated capacity). CDM introduced a flushing demand of 8,400 gpm at the intersection of Raeford Road and Reilly Road. Under this scenario, the model indicated that pressures begin to become less than optimal in the western portion of the distribution system, but still above 30 psi. The model also indicated that PWC would not be able to maintain the Cliffdale elevated storage tank at the 370-foot grade line, but would decrease its operating level to 367 feet, and thus would have less operating range and volume. The model indicated that flow from unflushed pipes into the flushing area does not occur.

Guidelines for Field Activities

During the course of designing the UDF program for PWC's distribution system, the following considerations and standards for field activities were developed:

- Effective and timely communications among all PWC staff. This is critical to the success of the UDF program. The staff includes engineering personnel, water resources personnel, construction personnel, WTF production personnel, GIS personnel, and flushing crews.
- Communications must be maintained with WTF personnel during flushing activities, particularly when large flushes are anticipated. Likewise, WTF personnel should give flushing supervisors notice of when they need to slow flushing activities to catch up with water production needs.
- In order to minimize flow reversal, the School Road repump facility should not be operated during flushing activities, unless absolutely necessary. Hydraulic modeling indicated that flows greater than 2.0 mgd from the School Road repump facility are likely to cause a flow reversal. Therefore, PWC established a standard of 1.5 mgd as the maximum flow from the facility on weekends during flushing activities.
- Flushing activities west of School Road will utilize the School Road repump facility, especially on the larger-diameter lines.
- To the maximum extent practical, the Glenville and Hoffer WTFs should meet water demand during flushing in order to maintain a consistent east-to-west flow direction.
- In order to flush the 24-in.-diameter mains along Raeford Road west of Skibo Road with water from the WTFs (without using the School Road repump facility), flushing must be performed during the night. This will minimize occurrences of low pressures in other parts of the distribution system.
- During drought conditions, special consideration should be given to flushing activities. Flushing activities may be limited to nighttime to conserve water and to mitigate potential public perception concerns.

- Flushing crews should maintain records, on the drawings, of any discrepancies between the drawings and the actual distribution system. Additionally, all valves or hydrants found to be in need of repair or replacement should be noted on the drawings and reported to engineering staff.
- All updates found in the field should be incorporated into the GIS database.
- After each loop, a member of the flushing crew should retrace the steps of the flushing loop to verify that valves are returned to their correct position. All valves, except certain strategic valves, are reopened. Strategically located valves that separate flushing zones should remain closed until the zone is flushed, in order to prevent mixing of flushed and unflushed waters.
- The following color code should be utilized during all distribution system activities, including UDF:
  - Pink dot = valve closed
  - Blue dot = valve returned to open
  - White dot = valve to be operated on upcoming loop

FLUSHING VELOCITY

As part of CDM's review of historical PWC flushing practices during the discolored water evaluation, it did not appear that velocity was being evaluated to check for the suggested velocity of 6 ft/sec. CDM recommended that flushing crews be provided with better guidance for appropriate flushing flows to be obtained for different-diameter mains and that a simple laminated table be provided to each flushing crew. Table 2 is an example of a flushing velocity table for various-size water distribution pipes. Using the orifice flow equation, the flow rate is determined based on the size of the hydrant opening and the recorded pressure reading.

Table 3 provides a summary of the lengths of pipe included in the flushing activities of these zones. This represents approximately 20% of PWC's distribution system.

Table 2 UDF flushing table to maintain 6 ft/sec

Main Pipe Diameter (*in.*)	Flow (*gpm*)	Pitot Pressure $2^1/_2$-in. Nozzle (*psi*)	Pitot Pressure $4^1/_2$-in. Nozzle (*psi*)
6	530	10	1
8	940	31	3
10	1,470	76	8
12	2,110	160	15
16	3,760	—	48
18	4,760	—	123
20	5,870	—	187

Table 3 Approximate length of pipes flushed in Phase 1 (Zones 3 and 4)

Main Diameter (*in.*)	Zone 3 Length Flushed (*ft*)	Zone 4 Length Flushed (*ft*)
6	289,000	152,000
8	299,000	157,000
10	10,300	0
12	53,500	59,100
16	71,400	4,000
20	1,400	0
24	21,000	18,000
Total	**745,600**	**390,100**

Total distribution system piping length is 1,100 miles.

CONCLUSIONS OF THE ENHANCED UDF PROGRAM

The UDF program has proven to be beneficial to PWC's water distribution system operation and maintenance program. PWC staff have indicated that the number of complaints of discolored water from customers in the Vanstory Hills, Montclair, and Haymont subdivisions has declined significantly since the enhanced UDF program was implemented, thus achieving the primary objective of this project. A description of other benefits observed follows:

- While developing the specific loop directions and maps, PWC staff found several locations at which the various infrastructure elements shown in the GIS were incomplete and/or incorrect. Therefore, during the activities required for developing the UDF flushing loops, PWC staff was also able to verify and update the PWC GIS database in several locations throughout the water distribution system. The project team developed a protocol for updating and correcting this information.
- During the course of developing the UDF loops, some neighborhoods and developments with a significant amount of customers were found to have only a single water feed line into the neighborhood. PWC has identified these areas and plans to provide dual feed into the areas where practical.
- PWC staff has located several valves within the distribution system that were partially or completely closed. Opening of these valves will improve the water quality, flow, and pressure available in those areas.
- PWC staff has greatly increased familiarity with the distribution system.
- The project further led to an understanding of the distribution system hydraulics and the impacts of various flow and pressure sources on the distribution system's flow patterns.
- The UDF activities removed a significant portion of material that causes discolored water from the distribution system.
- PWC staff increased public awareness of PWC's proactive approach to maximizing water quality and customer service. At least one article in the *Fayetteville Observer* provided an opportunity to answer a question regarding the UDF program in a positive and educational manner.
- The project established a baseline for further development of the UDF program throughout the rest of PWC's distribution system.

CHAPTER · 44

Mains Flushing and Sediment Measurement in Water Mains

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SUMMARY

This chapter describes two new measurement techniques that are in use in the UK to help make informed decisions about the need for mains maintenance to improve conveyed water quality to customers. Examples of their use are also provided.

INTRODUCTION

Water mains rehabilitation has formed a major portion of the capital expenditure within the UK water supply industry in the past decade, to overcome a backlog of deteriorating unlined cast-iron mains. Alongside this there has been an ongoing program of improvements to treated water quality. Both of these initiatives have led to an improvement in tap water quality. While capital expenditure on mains and treatment works has increased, in general there has been a decrease in regular maintenance activities, in particular, mains flushing. With new mains and good water quality, it could be argued that the need for maintenance is less pressing now than in the past. Nonetheless, the need still remains to maintain good water quality at the customer's tap. A further change that has occurred within distribution systems within the last decade is the formation of district meter areas (DMAs) and major reductions in leakage levels. With the formation of DMAs comes the introduction of dead ends (on former through mains) and closed systems, and improved leakage levels will lead to less natural "flushing" of distribution systems. Both could potentially increase sediment buildup in mains and increase the need for maintenance.

Rehabilitation and maintenance of distribution networks are expensive. Identifying which pipes require remedial action is not straightforward. There are many reasons why remedial action is required, amongst them, to improve the conveyed water quality to customers. Deterioration of water quality while in transit through distribution pipework occurs for two main reasons, namely corrosion of unlined iron pipework, and disturbance of sediment, originating either from the treated water or from disturbance of corrosion products. Both causes have the potential to lead to "red water" and other problems. Current measurement techniques to establish the requirements for remedial action include water sampling and pipe sampling, plus relying on data such as customer complaints. A larger choice of measurement techniques could help in making better informed decisions about mains maintenance.

It is clear that under certain circumstances, mains maintenance is necessary to maintain or improve conveyed water quality to customers. Intuitively, it could be predicted that the risk of sediment buildup would be highest, for example, at dead ends, in unlined mains, where water velocities are low and where treated water quality is poor. It would thus be possible to formulate a priority list of DMAs for maintenance based on factors such as treated water and distributed water quality, percentage of iron mains, and modelled water velocities. Other factors, such as customer complaints, leakage levels, and bursts, could also be included. However, very little documented evidence exists to confirm what the factors are that govern sediment buildup in mains.

This chapter describes two new measurement techniques that provide information on both large- and small-diameter mains in the system. The first technique involves the use of a new instrument, the "trunk mains sampler," which has been designed to sample deposits from inside trunk mains, under pressure, without interruptions to supply and without causing discolored water problems for customers. The purpose of the sampler is to provide objective measurements of deposits, from which it is possible, together with additional information such as conveyed water quality, to ascertain priorities for trunk mains rehabilitation, typically for mains greater than 6 in. A camera, which can also be inserted into trunk mains under pressure, has been developed and is used to provide visual confirmation of the presence of sediment and the condition of the trunk mains. A description of the development work, the operation of the sampler, and the interpretation of the results is given. Case studies of recent applications of the sampler to determine mains rehabilitation requirements are also provided.

The chapter will also describe a sampling method and results from a project aimed at establishing the factors affecting the buildup of sediment in small-diameter distribution mains. The objective of the project was to establish ground rules and trends to predict the risk of deposit buildup within distribution systems to help target maintenance flushing of small-diameter mains and swabbing of larger-diameter mains.

TRUNK MAINS SAMPLER

Development and Procedure

The development of the trunk mains sampler* commenced in 1995, when a need was identified to establish a main's internal condition without cutting into the main and disrupting supply to customers. A period of development followed that resulted in the design and manufacture of two items of equipment. The first is the trunk mains sampler itself, shown in use in Figure 1. The sampler is designed to measure sediment depth in water mains and to collect a sample of the sediment for subsequent chemical analysis. Mains from 4 in. to 48 in. in diameter and 1–20 bar pressure can be inspected.

The trunk mains sampler typically utilizes 2-in. gate valves installed on water mains (an example of which is shown in Figure 2), which allows internal inspection of underpressure trunk mains, without interruption of supply. A variety of other valve arrangements have been used.

The procedure followed on-site is described as follows. The trunk main sampler is attached to the tapping point and the gate valve is opened. The sample head is moved past the gate valve and down into the main using a battery-operated drill via a gearbox mechanism. Once through the valve, the head is "drilled" down to near the invert, where a spring mechanism trigger hits the pipe and the sampler head springs down onto the pipe. The head is then fully drilled down to achieve a seal on the pipe. The sampler head encloses a known area of trunk main (about an inch in diameter) and traps the sediment within that area. The sample tap is opened, which allows a controlled flow of water to resuspend the deposit, which travels through the hollow tube of the central sampler shaft and is collected in a sample bottle. The sampler head is then retracted fully from the main, the gate valve is shut, and the sampler is removed.

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* The patent for the trunk mains sampler is held by United Utilities in the UK. The license to use the sampler is held by Mayfield Consulting Ltd.

Figure 1 The trunk mains sampler in position on a trunk main

The sediment samples are taken to the laboratory where they are color coded using a standard color chart[*] to grade the discoloration of the water and the sediment sample. The option for analysis of the samples is available to establish the metal constituents, and typically this could be for the proportion by weight of iron, aluminum, and manganese. The remainder of the samples then undergoes analysis for the volume (and hence the depth) of sediment.

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[*] The standard color chart is similar to a paint chart with 10 colored squares, colored from white through yellow, to orange and black, intended to represent shades of discolored water. Samples can thus be graded from 0 to 9, with 0 representing clear water and 9 representing black water.

Figure 2 A typical chamber with a 2-in. gate valve installed for the inspection using the trunk mains sampler

The second piece of equipment developed was a camera and camera insertion device, designed to obtain a visual picture of the internal pipe condition, again without interrupting the supply of water to customers. The system for gaining entry to the trunk main is similar for both the trunk mains sampler and the camera.

On-site, the camera housing is attached to the tapping point. The camera and light source are fed into the housing and the video recorder is started. The gate valve can be seen to be opening, and the camera's approach to the pipe invert can be seen as it is "drilled" down in the same way as the sampler. Video footage looking directly down toward the bottom of the pipe is thus obtained. Similarly, as the camera housing is retracted, its progress can clearly be seen. Once the gate valve is shut, the camera and housing may be removed and the recorder switched off. Figure 3 shows a typical still picture of the internal pipe surface, taken using the video camera. The field of view in Figure 3 is approximately 6 in. by 8 in.

The camera has been further developed such that it can also provide a view of both the invert and the side walls of the pipe. It has also been

Figure 3 Photo of the internal surface of a corroded iron pipe showing scattered corrosion nodules

adapted for use to establish the absence or presence of sediments in service reservoirs.

For each site the results presented include a description of the sediment, an optional analysis of its metal constituents (Fe, Mn, and Al), a sediment depth, and a color code (to indicate its relative ability to discolor the water). Also presented is a description of the internal surface of the main directly under the tapping point, plus a still frame of the pipe invert.

Over the past seven years, the trunk main sampler has been used to inspect the internal condition of over 700 trunk mains in the UK, to assist water companies in deciding on rehabilitation requirements for the mains.

Results From the Use of the Trunk Mains Sampler

Assessments of individual mains

The trunk mains sampler results are typically used to establish rehabilitation requirements for individual lengths of main—usually those that are expensive and disruptive to take out of service in order to take pipe samples. A short report is produced, containing a table similar to that reproduced in Table 1. Photos of the internal pipe surface and a video are

Table 1 Example results from trunk mains sampler inspections

Site Ref	Description of Sediment and From Photo	Percent			CG*	Sediment Depth (*mm*)
		Fe	Al	Mn		
Site 1	**Sediment**: Dark brown deposits and large black flakes of bitumen. No rust. **Camera**: Large number of bits in the water. Badly tuberculated main covered in a layer of dark deposits. Coupon visible. Side view: Large corrosion nodules.	67	17	16	4	2
Site 2	**Sediment**: Dark brown floccular deposits and a few grains of sand. **Camera**: Lots of bits in the water. Dark stained cement-lined main with thin coating of dark deposits. Side view: Stained cement-lined main.	72	15	13	2	0.2
Site 3	**Sediment**: Dark orangey/brown deposits and large pieces of rust. **Camera**: Badly corroded coupon visible. Corroded main with large corrosion nodules and covered with a brown floccular deposit. Side view: Large corrosion nodules at top and sides of pipe.	86	7	7	4	3.6

*CG = Color Grade, graded from 0 to 9 against a standard color chart, with 0 representing clear water and 9 representing black water contained within the report. Recommendations for rehabilitation of the main are usually listed.

Overview of internal condition of trunk mains

In order to assess the factors that affect the buildup of sediment and corrosion within trunk mains, an analysis has been performed of sediment depths and internal condition for all 700 internal inspections of trunk mains. The trunk mains were inspected using the trunk mains sampler, which provides a sediment depth, a color grade of the 1-L water sample containing the sediment, and a video picture of the internal surface at the inspection point.

All the inspections have been categorized by unlined iron and lined or non-iron, hard or soft water and good or poor conveyed water quality. Of the 703 inspections, three quarters were in hard water, and 59% were on unlined iron mains.

Figure 4 shows a graph of the distribution of sediment depths for all sample points and the percentage of the various sediment depth bands subdivided by iron and non-iron mains.

The following observations can be made from the graphs:

- Only one in five trunk mains inspected had sediment depths above 3 mm.
- Two thirds of the trunk mains had less than 1 mm of sediment.
- The vast majority of high sediment depths (four out of five) were in unlined iron mains.

Figure 5 shows percentages of iron mains within each corrosion category from the inspections using the video camera.

This shows that almost half (44%) of the mains inspected were badly tuberculated, with a complete covering of corrosion nodules. It should be noted that the majority of the inspections performed to date have been on the mains that were being considered for rehabilitation. Thus it is not surprising that such a high proportion were in poor internal condition. The proportion of poor-condition trunk mains within the UK is likely to be considerably lower.

Figure 6 shows sediment depths for the non-iron mains only, categorized by the quality of the conveyed water. This shows that only 7% of non-iron mains had sediment depths above 3 mm. As would be expected, the higher sediment depths occur predominantly when conveyed water quality is poor.

Thus the analysis of the trunk mains sampling data has shown that:

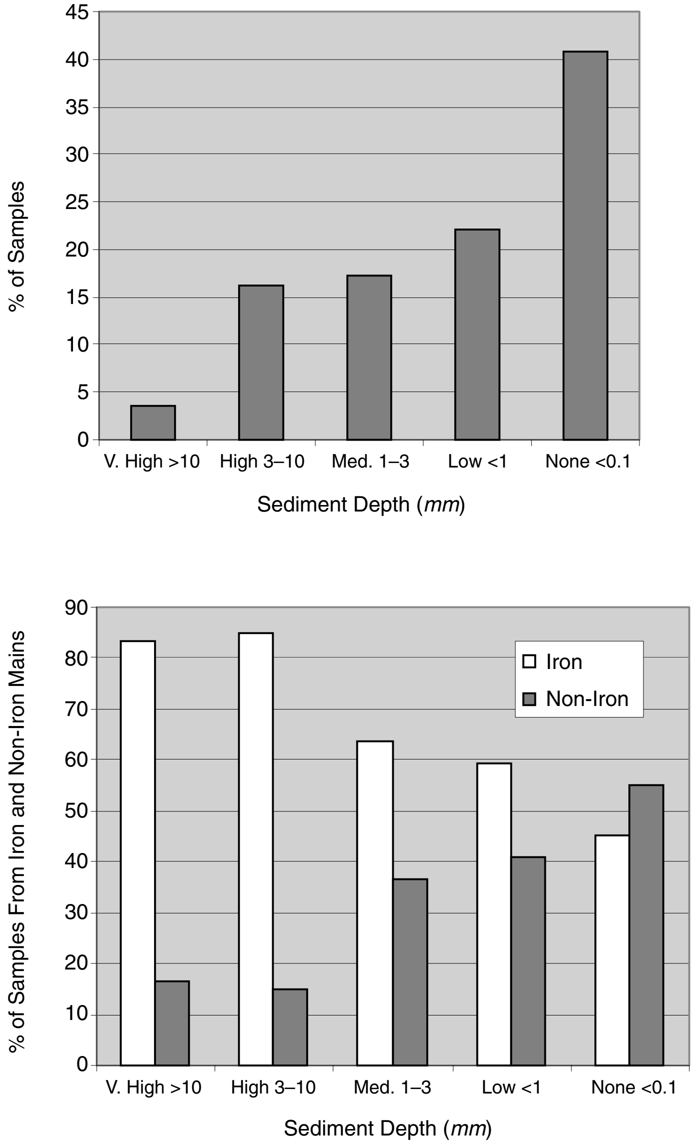


Figure 4 Distribution of sediment depths by mains material

- Eighty percent of the trunk mains inspected had sediment depths of less than 3 mm.
- Sediment depths were higher when conveyed water quality was poor.

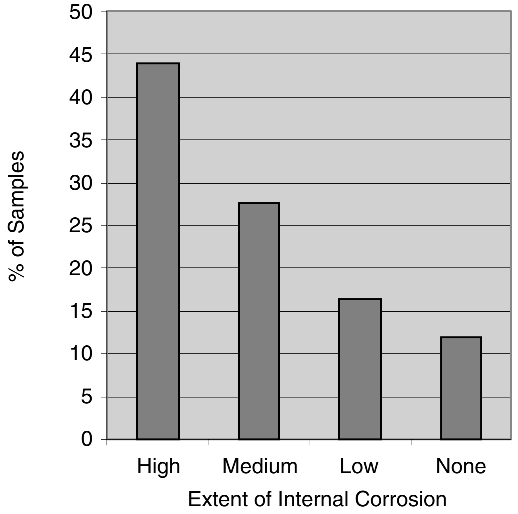


Figure 5 Percentage of samples from iron mains within different corrosion categories

- Just under half of the unlined iron trunk mains inspected were badly corroded.
- Corrosion was similar in hard and soft water areas.

STANDARD HYDRANT FLUSHING AND HYDRANT COLOR GRADE MEASUREMENT

Measurement Method and Flushing Procedure

Mains flushing is a standard maintenance technique for removing deposits from water mains. It is a labor-intensive and hence costly occupation. It is not normally practical or necessary to flush all the mains in an entire region. There is therefore a need to establish which mains contain the most sediment and which contain the least, to target those most in need of flushing. In order to identify mains with and without sediment, a standard technique was required that provided a simple objective measure of sediment quantity. In order for such a technique to produce comparable results, a standard flushing procedure must be followed. The sediment quantity measurement method, together with the standard hydrant flushing procedure that has been developed, is

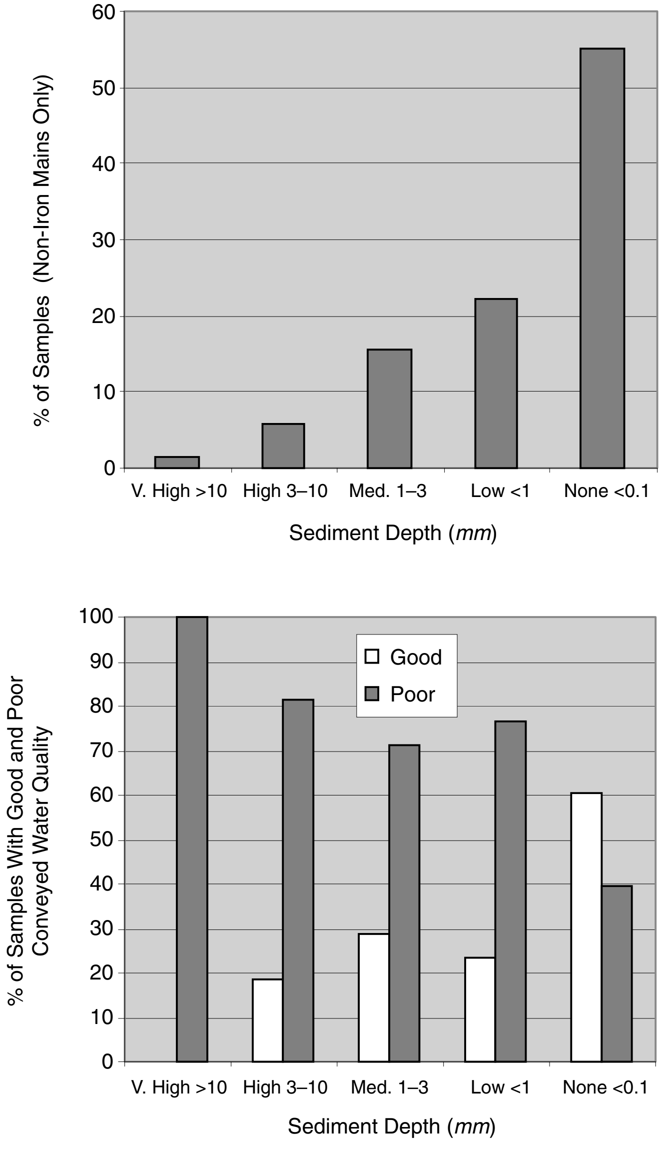


Figure 6 Percentage of samples with different sediment depths, categorized by conveyed water quality

described in Appendix A on page 915. The benefits of the technique are that it is simple to implement under field conditions and it requires no expensive equipment.

Results presented below are from a project funded by 13 UK water companies, the objective of which was to establish the factors that had the greatest effect on sediment buildup. A set hydrant flushing and measurement procedure was used and a standard form was completed. The procedure involved flushing the hydrant at a set flow rate, dependent on main diameter, to ensure sufficient water velocity to suspend and remove particulate matter. The measurement technique was simple and straightforward and involved taking water samples from the hydrant at one-minute intervals for six minutes and color-coding the resultant sample against a standard color chart. From this a total color grade could be calculated, which can be used as an objective comparative measure of the sediment quantity within the mains. The time taken for the water to run clear was also noted on the form, as were main material, lining, diameter, and whether or not it was a dead-end hydrant. Contributing water companies performed hydrant flushing within their areas using the set procedures and provided the data for analysis as part of the project. Subsidiary information, such as treated water quality and water type, water hardness, customer complaint numbers, and DMA population, was requested and collated.

Hydrant Flushing Sediment Measurement Data Analysis

Overview: Complete data set

Hydrant flushing data were received from 1989 hydrants, from 79 DMAs within 10 water companies. Just over half (59%) of the hydrants flushed were dead-end hydrants, and just over half (56%) were from unlined iron mains. The average total color grade was 14, and the average time to clear was 12 minutes. To give some idea of scale, a total color grade or time to clear of less than 5 represents a relatively clean hydrant, whereas if both measurements are above 20, the hydrant would be considered dirty. Table 2 provides the average total color grade and the average time to clear by main position and main material respectively.

Intuitively, it could be predicted that sediment buildup would be worse within the unlined mains and at long dead ends, and that time to clear would be longest under these two circumstances. The pooled data from all zones and all water companies do not confirm the prediction (with the exception of total color grade being worst at dead ends), indicating that other factors influence the buildup of sediment.

Figures 7 and 8 show the distribution of total color grade and time to clear for iron and non-iron mains and for dead-end and non-dead-end mains. Again, there is no indication of iron mains or dead-end mains

Table 2 Average total color grade and average time to clear, categorized by main position and main material

Position or Material	Average Total Color Grade	Average Time to Clear
Dead end	15	10
Middle	13	11
Through	11	13
Fe	14	11
Non-Fe	14	14
Total	14	12

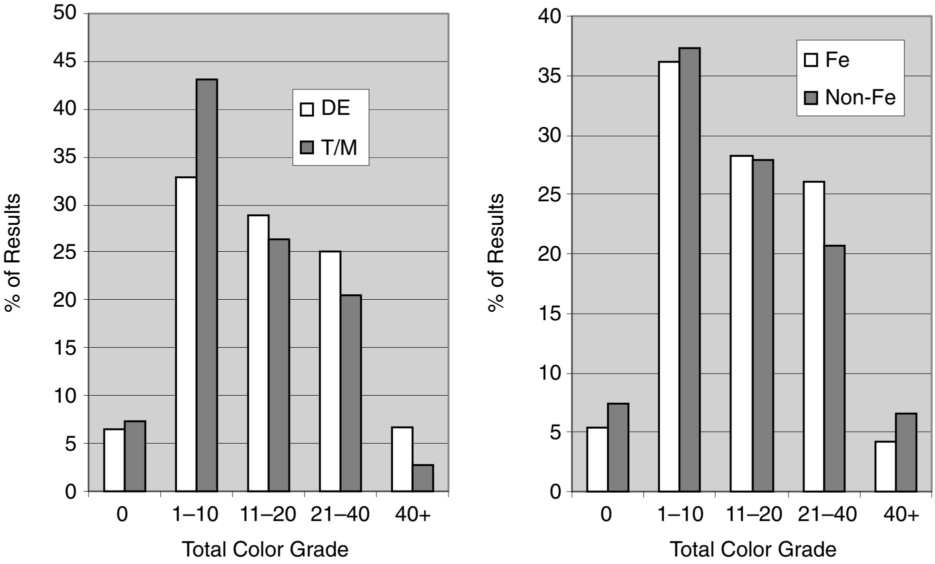


Figure 7 Distribution of total color grade, by main position and material

containing more sediment, or of taking longer to clear than non-iron or non-dead-end mains.

From the figures it can be seen that about three quarters (70%) of mains flushed, cleared within 10 minutes, and a third (34%) of mains had total color grades of less than 10. One in 10 of the hydrants took over 20 minutes to run clear, and over a quarter (28%) had total color grades

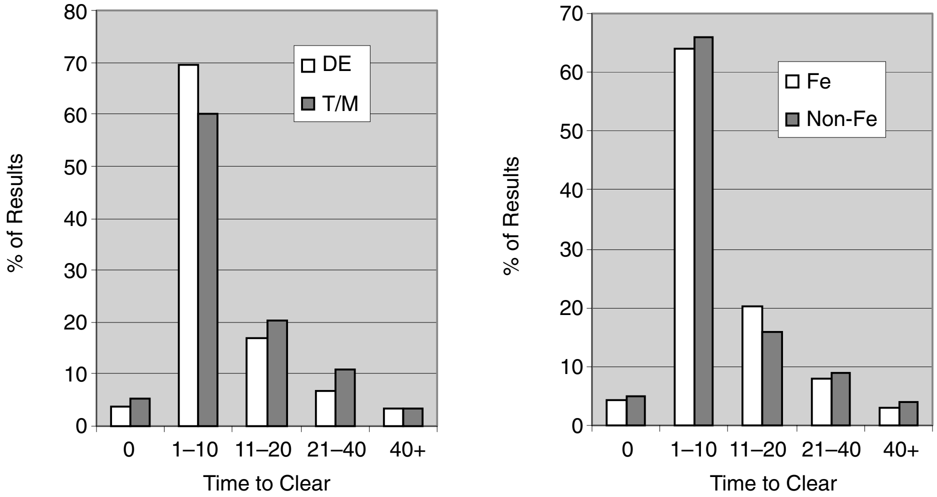


Figure 8 Distribution of time to clear, by main position and material

above 20. This provides an overview of the range of results that could be expected from using this simple monitoring technique, which clearly provides a simple method of differentiating between clean and dirty mains and a means by which to judge whether improvement in sediment buildup has arisen as a result of flushing.

Factors affecting buildup of sediment

The average total color grade and the average time to clear were calculated for various combinations of coagulant, treated water quality, water hardness, water type, and main material. Table 3 summarizes the trends evident from the analysis. The results indicate that, as expected, sediment depths and time to clear are highest where poor water quality is supplied and where aluminum coagulation forms part of the treatment process. Unexpectedly, the medium, rather than the soft water, had the most sediment; the upland, rather than the groundwater, had the least sediment; and, most surprisingly, the non-iron mains had the most sediment and the unlined mains the least.

As part of the project, data were collected to establish whether there was any correlation between sediment buildup and customer complaint levels. The data analysis showed that there was a general trend toward increasing customer complaints as the average time to clear increased.

Table 3 Summary of observations from an analysis of the variation of sediment quantities with various factors

Variable	Least Sediment	Most Sediment	As Expected
Coagulant	No coagulant	Aluminum	Y
Treated water quality	Good	Poor	Y
Water hardness	Hard & soft	Medium	N
Water type	Upland	Surface	N
Mains material	Iron	Non-iron	N

Data were also collected to establish whether customer complaints and measured water quality improved following mains flushing. The results indicated that an improvement in both these parameters was not guaranteed, and in fact rarely occurred.

Having tabulated the data, a statistical analysis was performed to establish the risk factors leading to sediment buildup. The outcome of the statistical analysis then needed to be translated into useful, pragmatic ground rules that will assist water companies in ranking their zones in priority order for mains flushing. The resulting ground rules are listed below. However, as the data did not provide clear-cut answers, it must be recognized that the rules listed below will not provide a perfect priority order each time. There is no substitute for a limited monitoring program to establish sediment quantities.

LOW PRIORITY. Groundwaters, hard waters, no coagulant, “good” treated water quality

HIGH PRIORITY. Soft water and long rural iron mains, “poor” treated water quality with aluminum treatment, high customer complaints

RESULTS AND CONCLUSIONS

Trunk Mains Sampler

The trunk mains sampler has been in use extensively within trunk mains in the UK to establish sediment depths and internal condition, and it has been shown that it can provide an accurate measure of sediment depth and a clear picture of internal condition. Analysis of the complete data set from all the trunk mains sampler inspections throughout the UK has shown that:

- Eighty percent of the trunk mains inspected had sediment depths of less than 3 mm.
- Sediment depths were higher when conveyed water quality was poor.
- Just under half of the unlined iron trunk mains inspected were badly corroded.
- Corrosion was similar in hard and soft water areas.

Hydrant Flushing Sediment Assessment

Data relating to sediment buildup and time for hydrants to produce clean water have been collected from almost 2,000 hydrants in 79 zones throughout the UK. Thirteen water companies worked together to produce the largest data set on sediment buildup collected in the UK. These data allowed conclusions to be drawn about factors affecting sediment buildup in distribution mains.

All mains studied contain some sediment. The risk of sediment buildup is highest in soft waters with long lengths of unlined mains, and generally such DMAs have high complaint frequencies. The risk is lowest in good-quality groundwater areas. However, no individual factor or combination of factors predicted the mains with high sediment buildup. The occurrence of high sediment buildup is unpredictable, making cost-effective targeting of mains flushing difficult.

A further finding was that flushing does not necessarily improve conveyed water quality during normal flow or lead to a reduction in customer complaints. Flushing reduces the impact of incidents. In badly corroded mains, flushing can cause discolored water. Mains rehabilitation may be a more suitable long-term solution in these cases.

Finally, the simple color grade measurement technique has proved successful for quantifying sediment in mains. Many UK water companies are using the measurement method during flushing operations.

Data collection is currently continuing, with a view to seeking further factors that affect the sediment buildup in distribution mains, and an updated report is due in 2003.

SIGNIFICANCE TO INDUSTRY

The two new measurement techniques described are in use in the UK to help in informed decision making about the need for mains maintenance to improve conveyed water quality to customers.

ACKNOWLEDGMENTS

The author would like to acknowledge the UK water company contributors for their cooperation during the field trials and for permission to publish this work. Thanks are also due to United Utilities (formerly North West Water), which holds the patent for the trunk mains sampler, and in particular to Mr. P. Cahill, who was the originator of the idea for the trunk mains sampler.

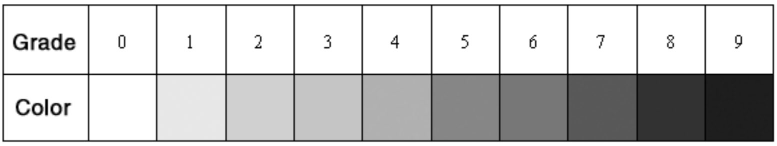
APPENDIX A: MONITORING PROCEDURE DURING MAINS FLUSHING

Perform hydrant flushing, following standard company procedures, with the flushing velocities listed below.

1. Number seven wide-necked bottles (preferably clear plastic), from 1 to 7.
2. Fill bottle 1 with hydrant water, approximately 30 seconds after opening the hydrant.
3. Take a further six samples at one-minute intervals after first opening the hydrant, into bottles 2 to 7, i.e., bottle 2 at 1 minute, bottle 3 at 2 minutes, etc.
4. Assign a color grade by shaking the bottles in turn and comparing the resultant color with the color chart below. Note the grades on the form. Also make a note of the visual appearance of the water and sediment by noting the color and particle description of the deposits in the final column (e.g., rusty orange water with large rust particles, black water with no visible particles, sandy deposits, and clear water, etc.).
5. When the water is clear and the flushing is about to stop, note the total flushing time to run clear on the form.

6. Discard the water and rinse the bottles so they are ready for use at the next hydrant.

Color Chart

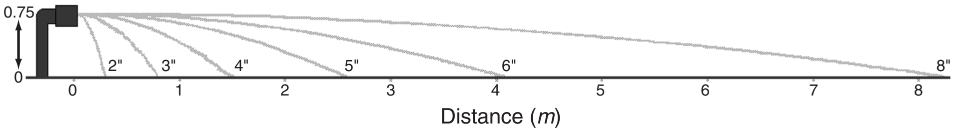


Grade	0	1	2	3	4	5	6	7	8	9
Color	□	░	░	░	▒	▒	▒	▓	█	█

Color image on page 1083.

Recommended flow rates and flushing velocities for various pipe diameters—to suspend loose deposits in water mains

Diameter		Flow	Velocity	Volume (m^3) From Flushing 100 m of Main Two Pipe Volumes	Time (*min*) to Flush 100 m of Main Two Pipe Volumes	Distance From Standpipe That Jet Hits the Ground, Using a 0.75 m Standpipe	
(*mm*)	(*in.*)	(*L/sec*)	(*m/sec*)			(*m*)	(*ft*)
50	2	1.5	0.76	0.4	4.4	0.30	1 ft
75	3	4	0.9	0.9	3.7	0.80	2 ft 6 in.
100	4	7.5	1.0	1.6	3.5	1.49	5 ft
125	5	13	1.1	2.5	3.1	2.59	8 ft 6 in.
150	6	20.5	1.2	3.5	2.9	4.08	13 ft 6 in.
200	8	41.5	1.3	6.3	2.5	8.27	27 ft



Trajectory of water at recommended flushing velocities for 2-in. to 8-in. mains

CHAPTER · 45

Distribution System Flushing Optimization to Reduce Environmental Impact

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Brian Murphy
Economic and Engineering Services, Inc., Bellevue, Washington

INTRODUCTION

The objective of this project was to optimize the Bellevue Utilities Department's (BUD) water main flushing program so that it effectively achieves BUD's goals of maintaining drinking water quality in accordance with the State of Washington Department of Health's standards while minimizing the impacts of distribution system flushing activities on receiving surface waters.

BUD is a wholesale customer of the City of Seattle and has little control over the quality of water that enters the drinking water distribution system. However, BUD does have full control over the water distribution system and the operational practices that can have a significant impact on water quality delivered to its customers. Since BUD has the opportunity to focus its attention and resources on the distribution system, its staff have developed monitoring and operation and maintenance practices that are designed to minimize deterioration of water quality through the distribution system while meeting other environmental goals of the utility.

One of the important maintenance tools implemented by BUD is a proactive water main flushing program designed to remove sediment and other materials from the drinking water distribution system. BUD conducts a unidirectional flushing program across the entire distribution system every four to five years. During flushing operations BUD field crews have targeted a flushing velocity of 7 feet per second (fps) or greater. BUD staff use the 7 fps velocity with the assumption that a velocity greater than operating conditions (2–5 fps) is needed to remove sediments. Flushing velocity also determines the volume of water for a given flushing period ultimately discharged.

BUD has a unidirectional flushing program that is state-of-the-art as far as potable water is concerned, but it may be possible to refine the existing program to better meet the requirements of more stringent environmental regulations and community expectations. BUD's primary source water, the Tolt River, was unfiltered until December 2000. The long detention times and low flow conditions possible in the water distribution system can be conducive to settling and accumulation of sediment in the pipes. The new filtration capabilities of the Tolt source, brought on-line in December 2000, may reduce but will not likely eliminate the need for a flushing program.

The primary outcome of this project was the revision of the BUD Zone Flushing Standard Operating Procedures (SOP) with respect to an optimum flushing velocity for BUD's water distribution system. The goal of the revised flushing velocity is to enable BUD water quality and operations personnel to achieve acceptable cleaning with minimum water volume and to minimize the impact of flushing activities on receiving surface waters directly attributable to flushing activities.

EXISTING FLUSHING PROGRAM AND PRACTICES

BUD implemented the current comprehensive unidirectional distribution flushing program in 1995 after recognizing trends in customer complaints of degraded taste. BUD observed difficulty in maintaining desired levels of disinfectant residuals in certain areas of the system and the accumulation of sediments originating from the Tolt water supply. Prior to 1995, BUD addressed customer complaints of diminished water quality by spot-flushing the affected area.

As a result of BUD's proactive flushing program, water quality was immediately improved. This is reflected by customer complaints and monitored chlorine residuals in one of the first flushed zones (Figures 1 and 2). In the long term, water quality continues to exceed the average quality attained with spot flushing. In 2001, BUD personnel began a second five-year flushing cycle. In December 2000 the new South Fork Tolt Filtration Facility was brought on-line; this facility is expected to reduce, but not eliminate, the need for distribution system flushing.

BUD operations staff maintain SOPs for both the flushing and dechlorination of flushed water. These SOPs are routinely reviewed and revised based on field experience in order to continually improve flushing practices.

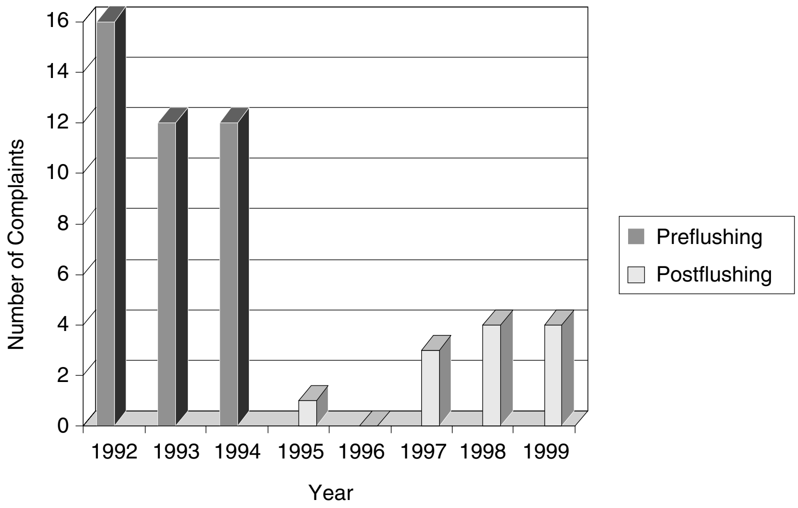

Figure 1 Customer water quality inquiries prior to and following unidirectional flushing of 335 zone

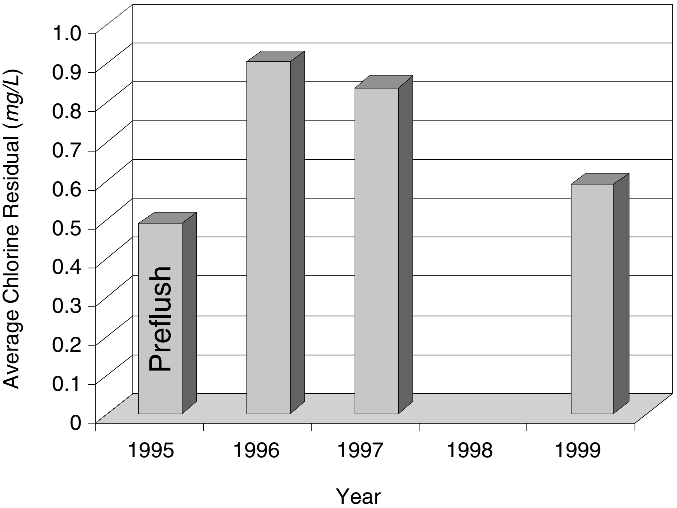

Figure 2 Distribution system annual average residual chlorine levels following unidirectional flushing of 335 zone

As a part of the flushing activity planning process, BUD crews review flushing discharge points to identify options for handling the water. Discharge receiving "facilities" are prioritized to eliminate or minimize potential surface water impact and include, in order of preference:

- Sanitary sewers
- Storm regional and/or neighborhood detention facilities
- Storm drain
- Direct discharge to surface waters

Three flushing sites were selected for impact analysis at storm drain outfalls. At each location, two to three flushing velocities were evaluated. All sites were flushed on December 19, 2000, under dry weather conditions. A summary of the flushing location details is shown in Table 1.

The velocities were established in an increasing step fashion. Sediments were collected from the hydrants at each velocity step to identify the material removed. Throughout the flushing process, water samples were collected at the outfall to the first catch basin affected by the flushing. Ambient water samples were collected when nonflushing baseline flows were present; samples were collected at 5-min intervals once flushing commenced. Sample analysis parameters and field data collected at each hydrant and outfall location are shown in Table 2.

The focus of this data analysis includes: (1) the impact of the flushing waters and the sediments transported from the distribution system, and (2) sediments resuspended in the storm drain and discharged to the receiving water. Storm drain discharge resulting from distribution system flushing consists of four components as it enters the receiving surface water:

- Total water volume
- Discharge intensity (cfs)
- Sediments originating from the distribution system,
- Resuspended sediments of the stormwater system

Based on historical practice, it was assumed that BUD does not discharge directly to surface streams. It was also assumed that energy dissipation occurs between the hydrant (due to diffusers and overland flow) and the storm drain point of entry so that the velocity of flow as it enters the storm drain system is essentially constant regardless of the hydrant flushing velocity. Therefore, velocity and duration of distribution system flushing affect the total volume of water discharged to the storm

Table 1 Flushing hydrant and storm drain outfall location parameters and flushing velocities

Hydrant Address	Hydrant Number	Pipe Diameter/ Material	Hydrant–Storm Drain Outfall Distance/Type	Targeted Flushing Velocities (*fps*)	Actual Velocity Recorded (*fps*)	Site Notes
10516 NE 20th Place	515	Combined 4-in. and 6-in. cast iron and AC	208 ft of 8-in. AC	2–4	$1.9^{*}/4.3^{\dagger}$	475 ft flushed consisting of 93 ft 6-in. cast iron and 382 ft of 4-in. AC pipe.
				4–6	$5.01^{*}/11.3^{\dagger}$	
				>6	$5.4^{*}/12.1^{\dagger}$	
2615 102nd Ave NE	490	6-in. AC	654 ft of 8-in. AC	2–4	4.0	600 ft flushed.
				4–6	4.7	
				>6	7.6	
10228 NE 20th Place	509	4-in. AC	421 ft of 8-in. AC	2–4	$8.6^{\ddagger}$	545 ft flushed. Hydrant sediment capture net burst.
				>6	$13.5^{\ddagger}$	

[*] Velocity on 6-in. run

[†] Velocity in 4-in. run

[‡] Velocities accidentally reached due to erroneous information indicating distribution line was 6-in. line

Table 2 Hydrant and outfall field and laboratory parameters

Hydrant Measured Parameters			Storm Drain Outfall Measured Parameters		
Parameter	Location	Method	Field	Location	Method
Temperature	Field	Std. Meth. 2550 B	Temperature	Field	Std. Meth. 2550 B.
Cl_2 residual	Field	Std. Meth. 500 Cl. G	pH	Field	Std. Meth. 4500-H^+B
pH	Field	Std. Meth. 4500-H^+B	TSS	Laboratory	EPA 160.2
Turbidity	Field	Std. Meth. 2130B	Settleable solids	Laboratory	EPA 160.5

drain. Further, velocity in the storm drain is a function of the parameters of the storm drain piping materials and design slope as governed by open-channel hydraulic principles and is beyond the control of BUD flushing crews.

All sediment and water quality data discussed in this analysis, unless otherwise noted, pertain to conditions at the storm drain outfall.

DISCHARGED SEDIMENTS

Sediments ultimately discharged to receiving waters are a combination of settleable solids and total suspended solids (TSS) originating in both the water distribution system and the storm drain system. The results of this fieldwork included: (1) analysis of the total discharge of sediments at the storm drain outfall regardless of point of origination; (2) a comparison of cumulative water volume discharge and cumulative sediment discharge; and (3) using reasonable assumptions, the differentiation and quantification of sediments from the water distribution system and the storm drain system.

Figures 3 through 5 show TSS and cumulative TSS at the receiving water outfall for each of the three study sites. Settleable solids are not shown in the figures. Review of the data showed that these solids typically contributed only a small fraction of total solids.

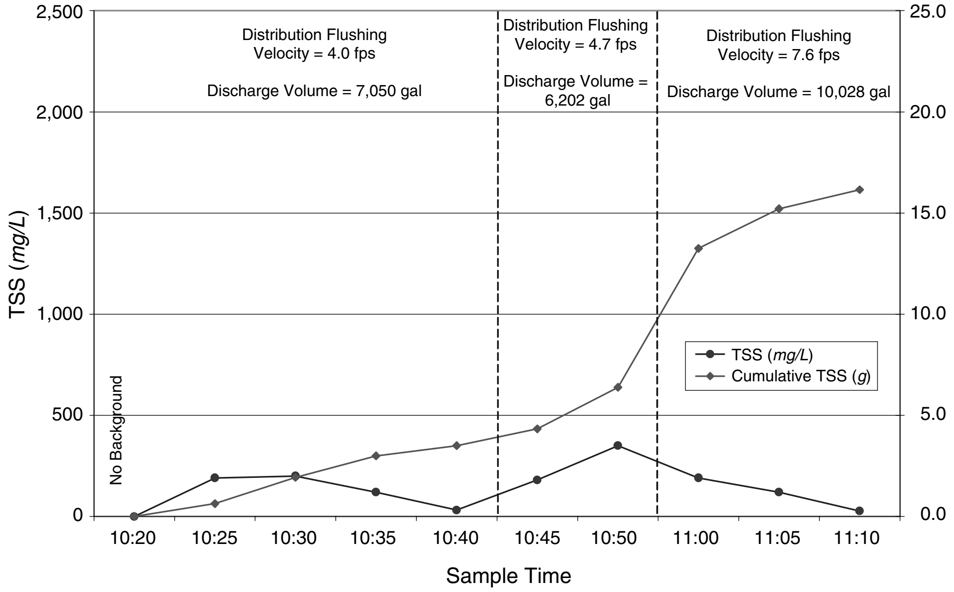

Figure 3 Sediment loading for outfall location 2615 102nd Avenue NE

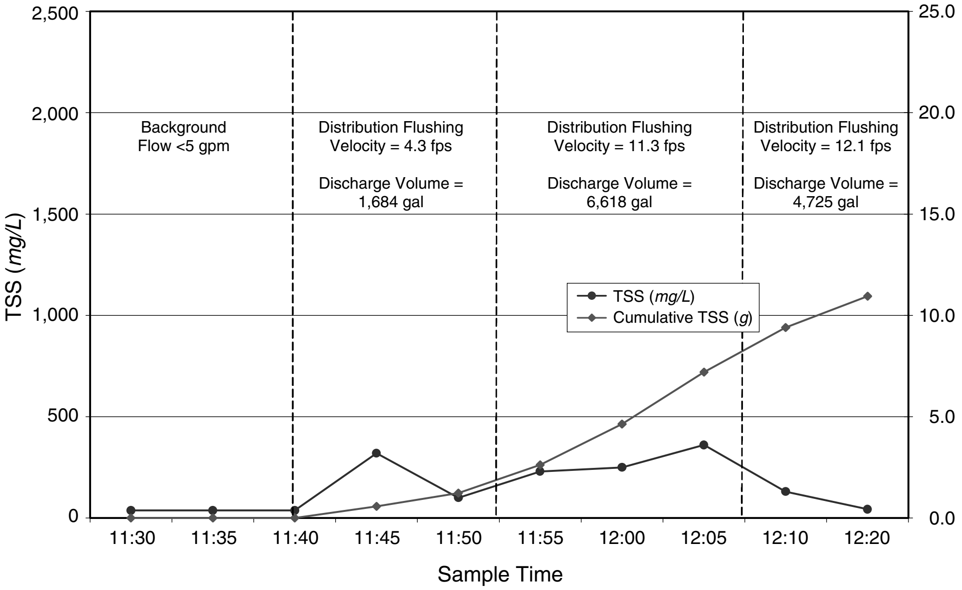

Figure 4 Sediment loading for outfall location 10516 NE 20th Place

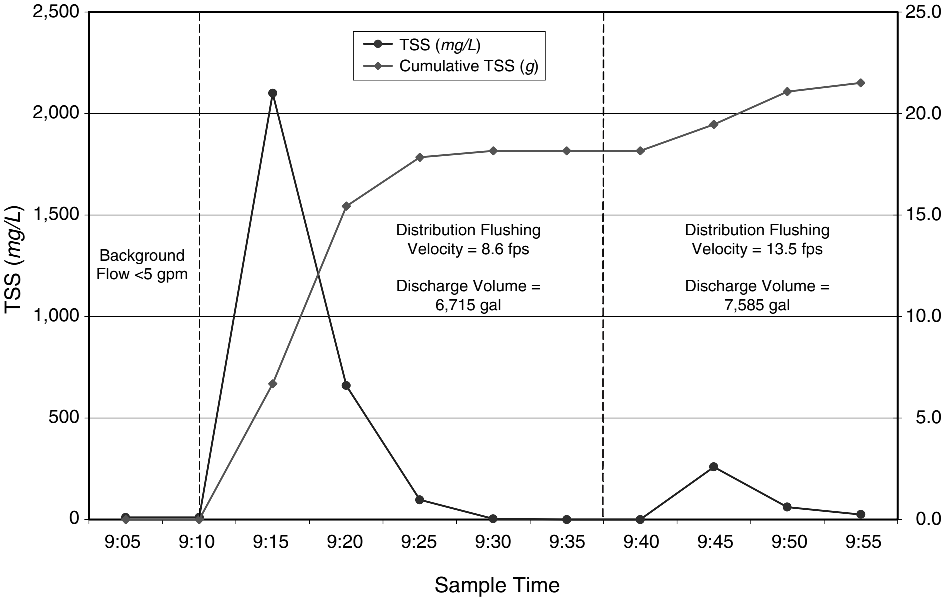

Figure 5 Sediment loading for outfall location 10228 NE 20th Place

Data collected from the 2615 102nd Avenue NE site and represented in Figure 3 are the best representation of the incrementally increased flushing velocity scenario. Velocities represented in Figure 4, while initially useful at 4.3 fps, later became much higher than would typically be used in the field. Figure 5 also is not representative of typical flushing conditions, as the initial flushing velocity is 8.6 fps and increases to 13.5 fps. However, while these data may not provide conclusive evidence of correlation of flushing velocity to types of sediment removed in the distribution system, they do reveal a pattern with regard to the disturbance of sediments in the storm drain system.

Each of the figures generally shows a spike in total suspended solids trailing a velocity change (an increase in volume). This would suggest that storm drain sediment disturbance is a function of flushing volume and the velocity created by the storm drain design.

Figures 3 and 4 indicate that sediment disturbance is more gradual when flushing is begun at a low velocity/volume and incrementally increased, whereas Figure 5 shows a rapid disturbance and discharge when flushing is initiated at a higher velocity/volume.

The distribution system sediments removed and captured at the hydrant as part of the fieldwork were within the size range of >57 to 4.75 mm. As expected, the larger percentage (by weight) of the sediments was skewed to the larger particles that intuitively would require a higher velocity to remove. Results of AwwaRF Project #2606 (Establishing Contaminant Specific Flushing Velocities) showed that it is not particle size but particle density that determines the necessary flushing velocity for removal. This finding is potentially important to BUD as the impacts of a new Seattle Public Utilities filtration plant on the North Fork Tolt River are realized. As the new plant continues to operate, it can be expected that future sediments in the distribution system, while reduced in quantity, will also shift the particle density distribution curve toward the finer, lower-density particles that theoretically can be removed with a lower velocity.

Figures 3 through 5 show total combined (distribution and storm drain) sediment discharges and do not distinguish among the origination points of the sediments. Sediments, debris, and other potential surface water pollutants can vary greatly from the drinking water distribution system and the storm drain system. Water distribution system debris generally consists of fine sediments to the occasional fist-size rocks that find their way into the system in a variety of ways. More important than the type of debris found in the distribution system is the quality of the debris in terms of potential detriment to surface water quality.

Distribution system debris, prior to flushing, is contained in a continuously disinfected, stable environment that is designed to be free of elements harmful to humans. The storm drain system, however, is designed to convey runoff water generated by finite storm events of varying intensity. As a result of handling this water, storm drains contain all surface contaminants that are carried into them by stormwater, including grease and oils, organic debris, fecal matter, sand, gravel, industrial chemicals, pesticides, herbicides, etc. Because of the quality disparity among these sediment sources, it is important to know their origin and quantity.

The concurrent AwwaRF Project #2606 (Establishing Contaminant Specific Flushing Velocities) field experiments were conducted with sediment-capturing “hydrant nets” designed to retain all particles larger than 57 μm. These nets were employed at both the 10228 NE 20th Place and 2615 102nd Avenue NE sites. For the purposes of this project, it is assumed that all sediments >57 μm originating in the distribution system were captured at the hydrant; therefore, all sediments reaching the storm drain outfall were considered to have originated in the storm drain.

Figure 6 shows a quantity comparison of sediment from each origination point at the two sites.

Based on a total mass comparison, the data clearly show that sediments entering the receiving water originate in the storm drain system and are transported by distribution system flushing activity. If it were assumed that all sediments would reach the receiving water in the absence of the capture nets, distribution system–originating sediments would constitute less than 2% of the total load.

In addition to the small contribution of distribution system sediments relative to the storm drain contribution, it is equally important to consider that flushing activities are the cause of storm drain sediment disturbance only once every five years. It is unlikely that sediments are being disturbed by flushing once every five years that would not otherwise be distributed by natural events like rainstorms that occur more frequently.

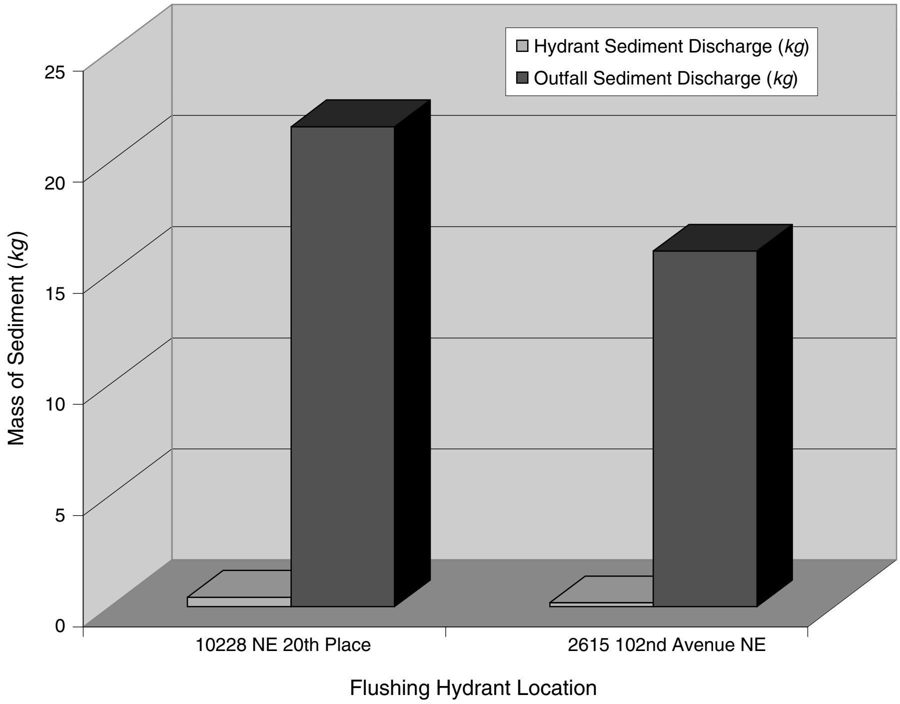

Figure 6 Sediment originating location comparison

Discharged sediment data were also evaluated to provide a comparison of cumulative water volume discharge, cumulative TSS discharge, and TSS concentration. These three parameters were examined to observe the relationship between water volume and sediment discharged. Figures 7 and 8 show a consistent level of sediment discharge while using a total amount of water approximately equal to or greater than that shown in Figure 9. Figure 9 shows a relatively rapid discharge of sediment (65% of the total in the first 5 minutes of flushing) compared with the other two locations where the largest single increase in sediment never exceeded 25% of the total. Based on these data, there does not appear to be a relationship between total water volume and sediment discharge, supporting the earlier hypothesis that the manner in which the water is applied is more of a factor.

Not only is the total sediment load at the end of the flushing activity of environmental interest, but the concentration of the sediment during the flushing activity is regulated under the Clean Water Act. Figures 7–9 show the concentration of sediment throughout the entire flushing period. The concentrations are cumulative in that they are calculated by dividing sediment load by cumulative water volume at each sampling time.

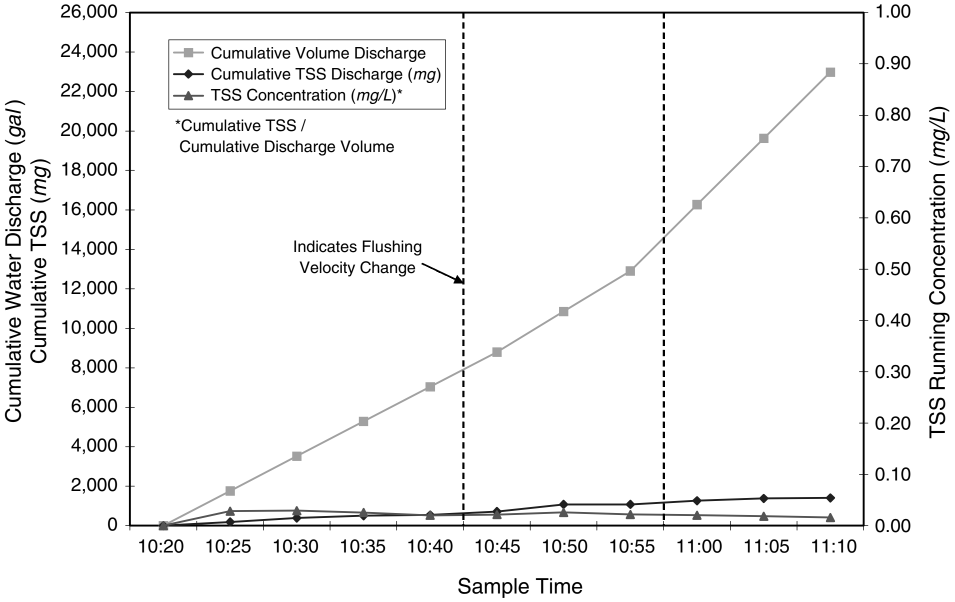

Figure 7 Comparison of cumulative volume and TSS discharge and running TSS concentration for 2615 102nd Avenue NE

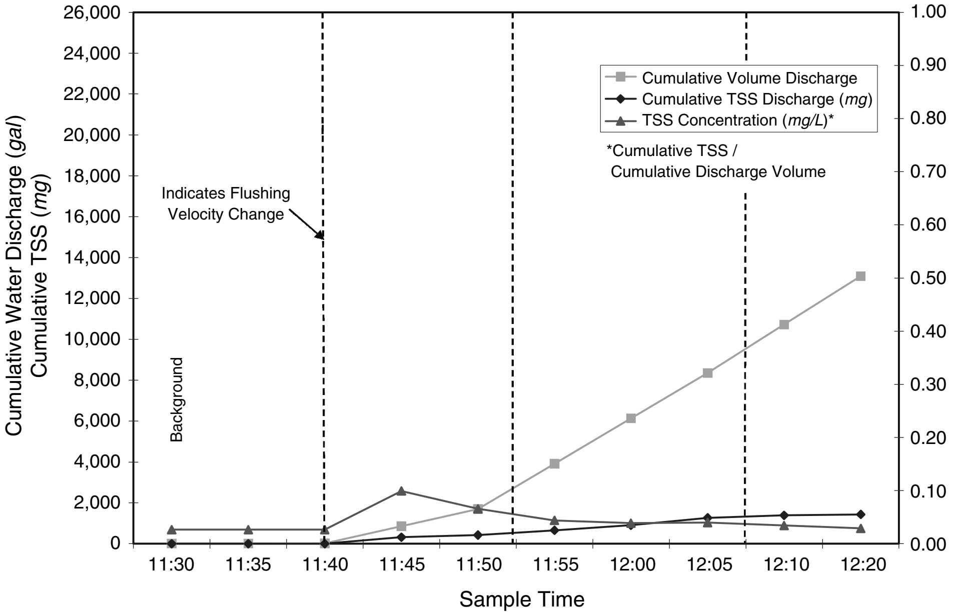

Figure 8 Comparison of cumulative volume and TSS discharge and running TSS concentration for 10516 NE 20th Place

Figure 7 shows a slight increase in concentration, which is unavoidable since no background flow was present, but then a slight decline. At no time did the calculated TSS concentration exceed 0.1 mg/L. Figures 8 and 9 are different from Figure 7 in that a background flow was present. Figure 8 shows an increase immediately after commencement of the flushing followed by a steady decrease to near-background levels at the end of the flush. The 10228 NE 20th Place site shown in Figure 9 was unique in that the introduction of flushing water actually provided a dilution mechanism and immediately reduced the concentration below background.

WATER VOLUME AND INTENSITY

The next two factors of environmental importance and related to stream habitat protection are the volume of water discharged and the period over which it is discharged (intensity). The impact of these parameters on a stream can vary depending on the background flow conditions of the stream. The discharge of flushing water may make up a greater percentage

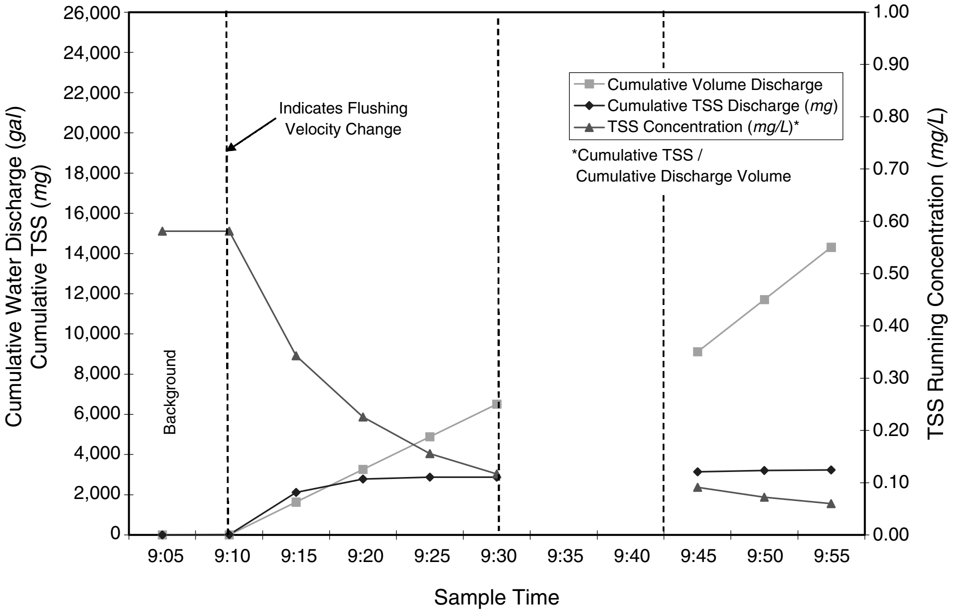

Figure 9 Comparison of cumulative volume and TSS discharge and running TSS concentration for 10228 NE 20th Place

of stream flow during drier periods than when streams are closer to capacity during and following wetter periods. The same holds true for intensity of the discharge.

Under normal operational protocol, BUD flushing crews currently target a flushing velocity of 7.0 fps and complete an average flush in 30 minutes. Assuming a 20% variability in flushing velocity, BUD crews are flushing in a range of 5.8–8.4 fps. This flushing velocity can be translated into the total discharge (gallons). A discharge intensity can be calculated by dividing the total discharge volume by the time period encompassed by the flush and converting units of gallons per minute to cubic feet per second. The estimated total water potentially discharged to a receiving water and the intensity of the flush, by pipe size, are summarized in Table 3.

Stream flows are a site-specific issue. Flows vary as expected not only by season, but also between streams due to drainage basin size and topography. Within the same stream, variance is due to location along the stream reach. Stream flows will be lower at the headwaters of a stream than near the lowland sections, where flow will be the greatest.

Table 3 Total estimated water volume (gallons) discharged and intensity during typical 30-min flushing operation

Pipe Size	Volumetric Discharge (*gal*) per Pipe Size for Typical Flushing Velocity Range			Discharge Intensity Range (*cfs*)
	5.8 fps	7.0 fps	8.4 fps	
4-in.	6,814	8,224	9,870	0.5–0.7
6-in.	15,333	18,505	22,206	1.1–1.6
8-in.	27,259	32,899	39,478	2.0–2.9

Revising the target flushing velocity could reduce the maximum volume of water discharged. Reducing the flushing velocity to 4.0 fps reduces the total discharge volume by 43% and reduces the intensity by approximately 38%. Table 4 summarizes the impact of reducing the target flushing velocity to 4.0 fps.

REVISED SOP FIELD TEST RESULTS

Field tests were conducted according to the revised SOP and the following field test recommendations:

- Test the new velocity at five locations, disposing of discharge through three “uncleaned” storm drain sections and through two recently “cleaned” storm drain sections.
- Test the new velocities in 6-in. pipe and 8-in. pipe.

The zone flushing SOP was field-tested at a total of five sites, as detailed in Table 5.

The objective of the revised SOP field tests was to show that (1) distribution system water quality objectives could be met at the lower velocity as measured by disinfectant residual and “time to clear” field observations, and (2) sediments resuspended in the stormwater system and discharged to the receiving water were decreased at the lower velocity.

Weather conditions were of importance during this round of sample collection. During collection at Sites 1–3 an intermittent light rain fell for 24 hr prior to collection, and a light rain was occurring during flushing

Table 4 Potential reductions in discharge volume and intensity resulting from reduced flushing velocity

	Volumetric Discharge (*gal*) for Given Flushing Velocity[*]				
Pipe Size	Existing 7.0 fps	Revised 4.0 fps	Volume Reduction (*gal*)	Revised Intensity (*cfs*)	Reduction From Existing Intensity[†] (*cfs*)
4-in.	8,224	4,700	3,524	0.26	0.35
6-in.	18,505	10,574	7,931	0.79	0.58
8-in.	32,899	18,799	15,000	1.40	1.04

[*] Assumes 30-min. flush time

[†] Assumes 7.0-fps initial velocity

activity. Precipitation was not heavy enough to create surface runoff conditions. Collection at Sites 4 and 5 occurred following 24 hr of light to heavy intermittent rain. Heavy rains began again midway through flushing at Site 4. Flushing and collection at Site 5 was delayed until storm runoff to the storm drain had ceased.

As shown in Table 5, flushing velocities ranged between 4.54 and 9.8 fps in application of the revised target velocity of 4.0 fps. The velocity at Sites 1, 2, and 3 was closer to the target velocity, ranging between 4.54 and 5.75 fps. Velocity at Sites 4 and 5 was recorded at 7.95 and 9.8 fps.

Figure 10 shows that the total suspended solids discharged using a target flushing velocity of 4.0 fps were reduced by approximately 3–16 kilograms. The data also show that the recorded time to clear was 25–30 min. The time to clear recorded for the two 7.0 fps target velocity sites was 35 and 45 min, but these data were collected under slightly different experimental conditions and are not directly comparable. However, the BUD flushing protocol at 7.0 fps typically requires a 30-min time to clear.

These data show that an equivalent time to clear can be achieved at 4.0 fps as at 7.0 fps. The data also indicate that less disturbance occurs in the storm drain pipe at the lower flushing velocity.

Flushing was also conducted in "clean" pipes to determine if cleaning storm drain sections prior to distribution system flushing would be beneficial. Based on the assumptions and the data presented in Figure 10, it was anticipated that such a practice would be beneficial.

Table 5 Revised SOP flushing hydrant and stormwater system specifications

Hydrant Address	Site ID	Flushed Pipe Diameter/Length	Storm Drain Outfall Length/Type	Targeted Flushing Velocity (*fps*)	Recorded Velocity (*fps*)	Site Notes
1. 8545 NE 12th St.	1	8 in. 1,090 ft	975 linear ft	4.0 fps	5.75 fps	Unclean
2. 8920 NE 13th St.	2	8 in. 1,552 ft	180 linear ft	4.0 fps	4.79 fps	Unclean
3. 8520 NE 15th St.	3	6 in. 1,465 ft	180 linear ft	4.0 fps	4.54 fps	Unclean
4. 106 Cedar Crest Ln.	4	6 in. 1,292 ft	8-in. concrete 574 linear ft	4.0 fps	9.8 fps	Clean
5. 1433 104th Ave. SE	5	6 in. 1,754 ft	8-in. concrete 476 linear ft	4.0 fps	7.95 fps	Clean

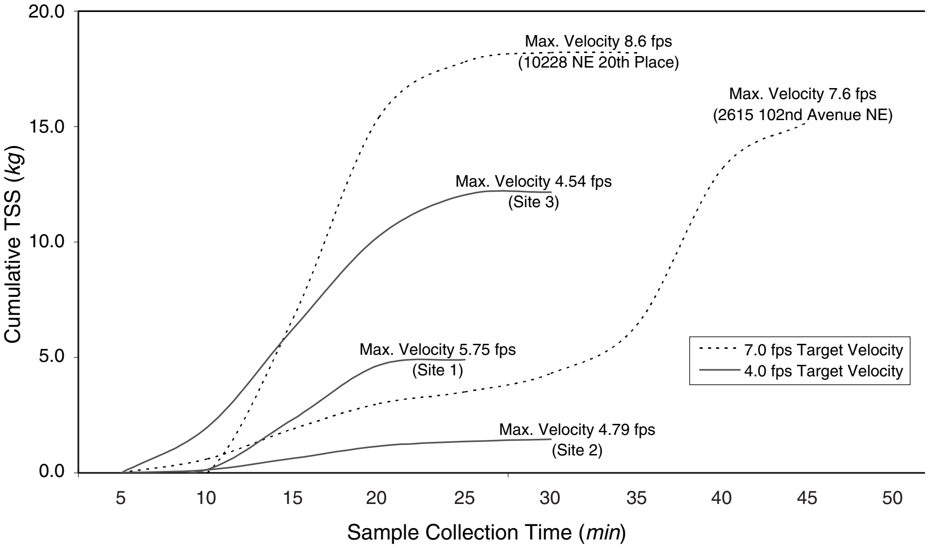

Figure 10 Comparison of cumulative total suspended solids at storm sewer outfall for 4.0 fps and 7.0 fps target flushing velocities

As a result of weather conditions, total suspended solids measured at the "clean" outfalls were comparable to those of the "unclean" outfalls. However, the data collected from "clean" outfalls were "contaminated" by heavy rain during flushing activities. These results have been attributed to the following:

- The storm drain lines were cleaned three weeks prior to flushing activities. Because this study was conducted during the wet season (the same period as typical flushing), redeposition of some sediments from storm events during the interim three weeks was possible.
- Possibly higher-velocity flow in the storm drain pipe due to reduced debris.
- More easily disturbed sediments due to recent deposition and disturbance.
- Heavy rains occurred during flushing, resulting in runoff to the storm drain at Site 5. Crews waited for flow into the stormwater pipe to diminish before beginning flushing.

- The "clean" lines were 2.5–3 times longer than two of the "dirty" pipes, making direct comparison difficult.

The total volume of flushing water, identified as a key factor in disturbance, was significantly reduced, as Table 6 shows. Applying a 4.0 fps flushing velocity reduced the volume of water by 67% in the 6-in. pipes and by 58% in the 8-in. pipes.

CONCLUSIONS AND RECOMMENDATIONS

It was the conclusion of this project that the impact of BUD's flushing program on surface waters does not outweigh the benefits. The BUD distribution system flushing program is a state-of-the-art unidirectional flushing program that maintains the distribution system and water quality by the most efficient and effective means currently available.

In an effort to limit environmental impact, BUD does not conduct flushing activities during the dry months of July and August. BUD flushing personnel conduct flushing and dechlorination activities as described in formalized SOP documents. Both SOPs contain procedures for evaluating and selecting a flushing approach to minimize environmental impact to the greatest extent possible. Both SOPs include procedures for evaluation and prioritization of flushing discharge options to limit potential impacts to natural environments.

Table 6 Theoretical and field-reported flushing water volumes

	Volumetric Discharge (*gal*) for Given Flushing Velocity*			
Pipe Size	Theoretical Calculated 7.0 fps	Theoretical Calculated 4.0 fps	Field-Reported Targeting 4.0 fps	Flushing Water Volume Reduction Over 7.0 fps Theoretical Volume (*gal*)
6-in.	18,505	10,574	8,000*	10,505
8-in.	32,899	18,799	13,875†	19,024

*Volume recorded from one of three 6-in. pipes. The other two pipes were flushed at velocities 2–2.5 times greater than target due to field conditions.

†Average of two 8-in. pipes flushed with a velocity of 4.79 and 5.75 fps

Based on the data collected during the initial and revised flushing velocity field tests, BUD water distribution system flushing activities do result in the discharge of suspended and settleable solids to receiving surface waters. The initial field results showed that virtually all of the discharged sediments (>98% by weight) originate in the storm drain system. This assumes that all sediments from the distribution system reach the receiving water. In practice, and depending on field conditions, it is likely that some of the distribution system sediments do not reach the storm drain system and receiving water.

The results of field tests conducted at a velocity of 4.0 fps showed that the time to clear was equivalent to the higher velocity of 7.0 fps. Field data showed that total water volumes were reduced by as much as 67% over the theoretical volumes calculated for the higher velocities. Actual reduction is variable and may often be greater.

The benefit of storm drain cleaning prior to flushing activities remains in question. It is intuitive that prior storm drain cleaning would reduce sediment discharge at the time of flushing. Based on the limited data of this field study and the variables associated with flushing activities, it would be reasonable to expect that any benefit would require close coordination between water and storm drain operations personnel. A cost-benefit analysis that considers the cost of coordination, the real benefit gained from such coordination, and how realistic such coordination is should be conducted at a management level at the utility.

A final observation of these field studies relates to achieving the target velocity. Field results show that target velocities were always exceeded, but in some cases they were exceeded by 2–2.5 times the target velocity. It is understandable that field conditions vary and application of a lower target velocity will require practice by field personnel. To focus the activity of field crews, an upper-limit target may be beneficial. A target range of ±20% of the 4-fps target would be appropriate.

CHAPTER · 46

Unidirectional Flushing: Operational Excellence at EPCOR Water Services

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SUMMARY

Unidirectional flushing (UDF) is an important program that EPCOR Water Services Inc. (EWSI) implements to ensure the distribution of high-quality water to its customers. This chapter describes the recent optimization study that compiled and analyzed seven years of data records to determine the most advantageous flushing frequencies for the city of Edmonton.

The 2002 study looked at optimizing the existing UDF Maintenance Program to decrease flushing frequencies while maintaining water quality and service. A great deal of information related to the effectiveness of UDF existed, so EWSI developed "Area Information Frequency Sheets." These sheets provided a systematic layout to compile all applicable data recorded since 1995. The area information includes:

- Basic statistics
- Flushing history
- Water quality
- Customer complaints

Initial conclusions indicated that Edmonton's existing two-year flushing frequency was more than adequate to maintain service of high water quality to EWSI's customers. The first conclusion of the study was encouraging, as it indicated that flushing frequencies could be decreased; however, it did not provide direction on how to divide the areas according to allowable time between flushings. From a literature review and the availability of Edmonton data, five factors or "triggers" were determined to be influential in governing the effect of unidirectional flushing.

The outcome of the optimization study was the assignment of a one- to four-year flushing cycle and the selection of either 2002 or 2003 as the next year to flush for each UDF area. This optimization study was a success. The budgeted number of UDF runs for 2002 was 2,450; the actual number of runs was 1,355. This reduced but effective flushing translated to a savings of approximately $150,000 Canadian in 2002. Over the next five years, the study projects a 25% reduction in annual UDF costs. Through the resulting efficient flushing, large cost savings will be realized through reduced manpower, energy consumption, and water use.

BACKGROUND

The condition of a distribution system can drastically affect the overall quality of potable water. Flushing is a useful pipe-cleaning technique that replaces poor-quality water with better-quality water and removes a portion of the sediments and biofilms. Implementing more efficient flushing techniques, like unidirectional flushing (UDF), can effectively improve both the water quality and the infrastructure of the distribution system by reducing the opportunity for biofilm growth and the accumulation of corrosion end products.

The city of Edmonton has performed conventional hydrant flushing since the 1960s. In the early 1980s, the program was discontinued for three years due to economic constraints but was reinstated to alleviate concerns about water quality standards. In 1990 a study was conducted to evaluate the effectiveness and economics of an alternative flushing method, unidirectional flushing (UDF). Its recommendations were executed in the 1991–1994 Unidirectional Hydrant Flushing Study. These studies demonstrated that UDF produced higher velocities and cleaned mains more effectively while using significantly less water than did conventional flushing. The conclusion of the pilot was to implement unidirectional flushing throughout Edmonton on a biennial frequency.

Presently, EPCOR Water Services Inc. (EWSI) divides approximately 3,100 km of the city of Edmonton's water distribution main into 4,851 UDF runs, forming 80 UDF areas. Figures 1 and 2 illustrate a UDF run and a UDF area respectively. Until 2002, EWSI ran Edmonton's UDF program based on a two-year flushing rotation of all city mains with the exception of Area 3156, the Goldbar/Capilano region, which was flushed annually. The program has been alternating flushing years between areas of mainly cast-iron (CI) pipe and areas of mainly asbestos–cement (AC) and polyvinyl chloride (PVC) pipe, approximately 2,500 UDF runs per year.

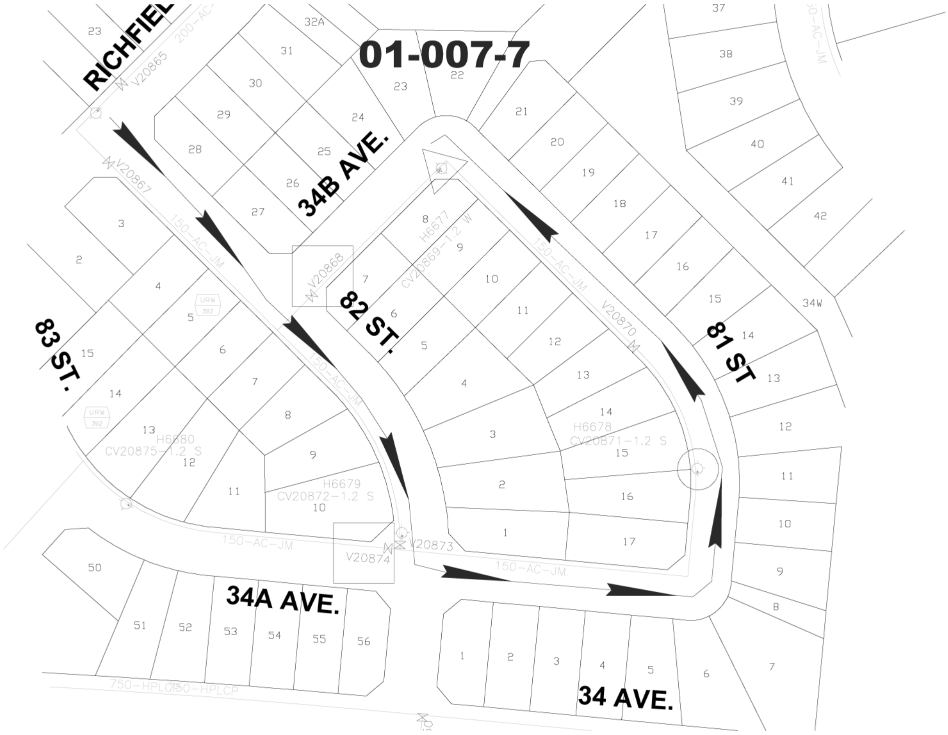

Figure 1 UDF run example

THE 2002 OPTIMIZATION STUDY

The first evaluation of the frequency of Edmonton's unidirectional flushing resulted in the UDF Optimization Study in 1999. One of the recommendations from the report was to incorporate baseline and post-flushing water quality samples, similar to the pilot program performed in Cincinnati (Cossins et al., 1999), in Edmonton's UDF program to provide an alternate method of evaluating the effectiveness of the flushing system. These pre- and postflushing samples were collected in 2000 and 2001.

The 2002 study took a close look at optimizing the flushing frequencies of EWSI's UDF Maintenance Program. The water quality sampling indicated that Edmonton's previous two-year flushing cycle was more than adequate to maintain service of high water quality to EWSI's customers. Reducing flushing would result in significant cost savings; however, in order to ensure water quality would not be compromised,

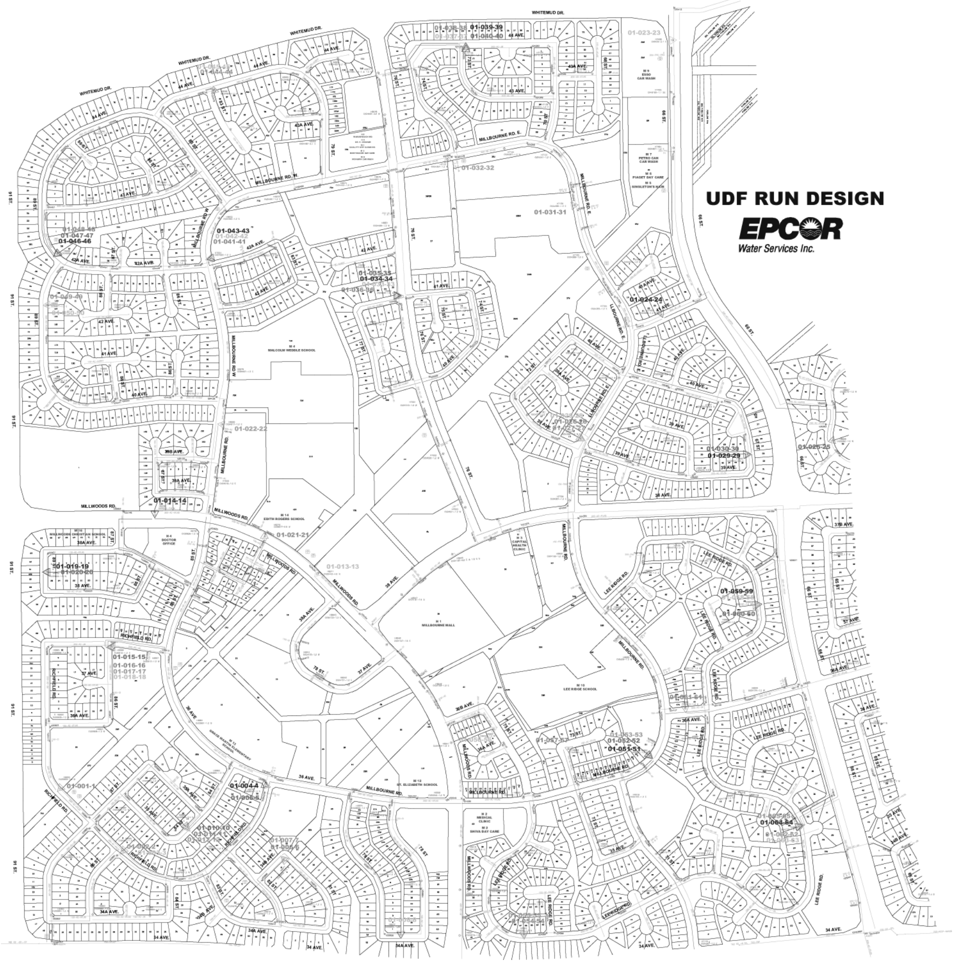

Figure 2 UDF area example

EWSI required intensive analysis before allowing changes to the successful Unidirectional Flushing Program.

DATA

The 2002 UDF Frequency Optimization Study started with the compilation of all data related to Edmonton's distribution system and UDF Maintenance Program, including the pre- and postflushing samples.

Abundant data existed, so EWSI developed "Area Information Frequency Sheets" to provide a systematic layout to compile all applicable data recorded since the initiation of the UDF Program in 1995. Table 1 summarizes the area information.

Five systems contribute to EWSI's data records for the Unidirectional Flushing Program:

- Unidirectional Flushing (UDF) Application
- Geographical Information System (GIS)
- Water Maintenance Management (WaMM)
- Hydraulic Model
- Laboratory Information Management System (LIMS)

The UDF application and GIS are evident systems involved in the UDF Maintenance Program. Together they generate the bridge between the field and the office through data forms and maps. Each year, the run designs are illustrated and the flushing results are recorded through these

Table 1 Summary of Edmonton's existing records

Information	Details
Basic statistics	• Number of runs, hydrants, and valves • Average length of runs and hydrant leads • Length of pipe by material
Flushing history	• Field records of flush time, start turbidity, and end turbidity • Calculated records of velocities achieved and water consumption
Water quality	Pre- and postflushing water samples • Turbidity • Disinfectant residual • Heterotrophic plate count
Customer water quality complaint	Six categories • Turbidity/color • Odor and taste • Hard water concerns • White particles • Health concerns • Other

documents. UDF and GIS systems are also essential for the design of the UDF runs. As well, the hydraulic model of the distribution system can be applied to confirm water flows and velocities to ensure an overall effective and efficient design. WaMM contains information significant for the actual practice of flushing. It supports the UDF Maintenance Program by ensuring the operability of the valves and hydrants or at least by identifying any out-of-service assets that may require runs to be redesigned. LIMS is a valuable tool to monitor the effectiveness of UDF. The water quality records for both sampling specific to the flushing and sampling based on customer complaints provide direction for problematic areas of the distribution system. Each system has an independent part to play in the design of unidirectional flushing runs; together the systems can be used to optimize the UDF Program.

ANALYSIS

The analysis process for the 2002 Optimization Study involved making decisions and highlighting significant factors from the vast amount of available data. EWSI had initially hoped the first conclusion would have provided unambiguous direction for the redesign of flushing frequencies. When it did not, EWSI conducted further study and research to choose other influential parameters from the existing data.

First Conclusion: Pre- and Postflushing Samples

The 2002 study looked at optimizing the current UDF Maintenance Program to decrease flushing frequencies while maintaining water quality and service. The first step was a statistical analysis of the pre- and postflushing samples. The 2000 sampling occurred in the areas of mostly CI; the 2001 sampling occurred in the areas of mostly AC and PVC. Baseline samples had been taken four weeks prior to the initiation of the UDF Program; postflushing samples had been taken four to six weeks after program completion. This extended time period separating the sampling and the flushing ensures that the system settles down after the UDF Program so the results accurately represent the water quality in the distribution system. Testing of these samples included turbidity, disinfectant residual, and heterotrophic plate count (HPC). The difference between the before and after results for each test was calculated for the overall results of the UDF areas.

The conclusion of this analysis in most cases was that after seven years of a consistent UDF pipe-cleaning program, further flushing made no significant improvement in the water quality. Actually, the trend for the disinfectant residual was that the levels had decreased in the few weeks after flushing. So the general conclusion of the pre- and postflushing samples indicated that Edmonton's two-year flushing cycle was more than adequate to maintain service of high water quality to EWSI's customers.

Second Conclusion: Five Triggers

The first conclusion of the 2002 study was encouraging, as it indicated that flushing frequencies could be decreased; however, it did not provide direction on how to divide the UDF areas according to allowable time between flushings. From a literature review and the availability of Edmonton data, five factors were determined to be influential in governing the effect of unidirectional flushing.

1. *Percent of total pipe length that is unlined cast iron*. Unlined cast-iron pipe has been correlated with lower water quality. The material has a high affinity for tubercles and sedimentation. In historical UDF field data, CI areas demonstrate higher starting turbidities as well as longer flushing times required for each run.
2. *Average length of hydrant leads*. Hydrant leads are locations of stagnant water in the distribution system. The longer the hydrant lead, the larger the volume of stagnant and likely poor-quality water.
3. *Average flushing time per run*. EWSI flushes all UDF runs until a maximum turbidity of 1 NTU is obtained. The crews will flush the run until they have discharged the volume of water contained in the length of run pipe and have reached the required turbidity.
4. *Field turbidity at start of flush.* Once the hydrant is opened, the field crew waits a few minutes to take the start-of-flush sample. Turbidity is a significant factor in water quality, and it is a quick and easy test to perform in the field.
5. *Number of total water quality complaints*. Both WaMM and LIMS maintain water quality complaint call records. These data were compiled from the two sources and organized into six categories based on the description of the call. The six categories were turbidity/color, taste/odor, white particles, hardness, illness, and others.

Individually, each of the triggers has weaknesses that reduce its ability to be the sole consideration in determining the flushing effectiveness. For example, the current procedure to measure starting turbidity is to wait a few minutes after opening the hydrant to take a sample. This period allows for most of the water sitting in the hydrant lead to be replaced with distribution water before sampling. Crews recorded extremely high turbidity values in 1995, possibly due to the condition of the water in the pipes or to sampling as soon as the hydrant was opened. Applying all five triggers should decrease the possibility of erroneously assigning a UDF area too long or short a duration before the next flush.

Balancing Act

Rank and percentile statistics were performed on each set of data for the most recent flushing year of each UDF area. As the initial concept was to develop a three-year flushing program, a percentile of 75% was chosen as the "trigger" point, in hopes that one third of the UDF runs would fall out from that and therefore balance these runs on a three-year basis.

The results, however, presented 42% of the UDF areas, equivalent to a third of the UDF runs, with all their triggers being under the 75th percentile. EWSI decided to assign these UDF areas a four-year flushing frequency, with the understanding that a monitoring program will be implemented to check the water quality and service within these areas on a regular basis. For the remaining UDF areas, EWSI recommended flushing frequencies according to the number of triggers above the 75th percentile and operational ability. The study further manipulated the flushing frequency and next year to flush for each UDF area in attempts to develop a schedule of equal number UDF runs for the next five years.

RESULTS

Information organization enabled an informed and confident decision. The 2002 Optimization Study produced a number of positive results. An indirect result, the simple development of the "Area Information Frequency Sheets," will improve the overall data management of the UDF Maintenance Program. The key result of the study, optimal unidirectional flushing frequencies, promotes a better balance among work effort, costs, high levels of water quality, and service to EWSI customers.

Area Information Sheets

The “Area Information Frequency Sheets” provided the ability to assimilate all of the influential records for unidirectional flushing. This sheet has now become the data management tool of the Unidirectional Flushing Program.

An example of the Area Information Frequency Sheet for one area in Edmonton is included in Appendix A. Area 3278 is now flushed annually, as indicated by all five of the triggers. It is located in the center of the city and therefore consists of older, unlined cast-iron pipes. However, hydraulics also contribute to the need to increase its flushing frequencies because its location is at a point where the flows from the two water treatment plants meet, creating a potential stagnant area.

Optimal Unidirectional Flushing Frequencies

The outcome of the 2002 Optimization Study was the assignment of a one- to four-year flushing cycle and the selection of either 2002 or 2003 as the next year to flush for each UDF area. Table 2 includes the number of UDF areas and runs in each frequency category. Figure 3 is a map of the city of Edmonton color-coded to display the resulting flushing frequency of all the UDF areas.

As stated previously, sampling indicated that EWSI’s current flushing frequencies were more than adequate to maintain service of high water quality in the majority of Edmonton’s distribution system. Increasing the time duration between flushing for 76% of the UDF areas reflects this. However, the optimization process identified three additional UDF areas requiring increased flushing frequency. Upon further analysis, these

Table 2 Frequency of Unidirectional Flushing Program

Frequency	Number of Areas	Number of Runs
1-year	4	375
2-year	15	1,125
3-year	28	1,811
4-year	33	1,540

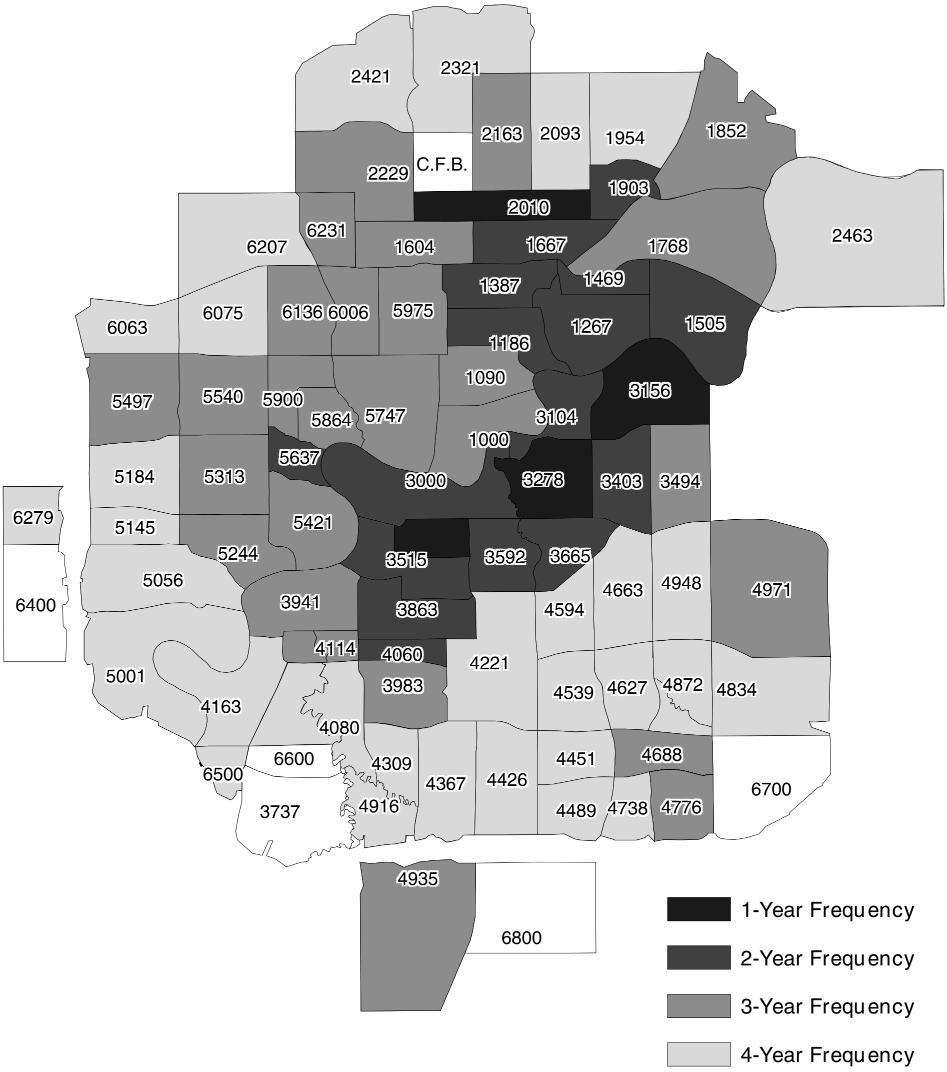

Figure 3 Map of frequency of Unidirectional Flushing Program

areas have demonstrated problems due to hydraulic issues such as influential pressure boundaries.

Edmonton's biennial UDF Maintenance Program flushed approximately 2,500 runs per year. Table 3 presents the five-year schedule resulting from the 2002 UDF Frequency Optimization Study. The recommended schedule is flexible and will be continually revised with additional flushing results, relining and cast-iron renewal projects, new growth areas, and general redesign of flushing areas. The suspension of flushing for up to four years will also occur in conjunction with a water quality monitoring program.

Cost Savings

The cost savings will be demonstrated through the field activity costs. Table 4 highlights the number of runs and cost savings resulting from the 2002 Optimization Study. Decreasing the number of UDF runs will decrease the number of field crews required and therefore decrease the operational cost for the program. However, it should be noted that the hydrant purging program will reflect an increased operational cost, as it will now include those hydrants no longer being inspected as part of the UDF Program.

In addition to the substantial operational cost savings, indirect savings will occur through the reduced use of water and power. Decreasing the number of UDF runs per year will decrease the amount of treated water flushed. This reduced water usage will reflect savings in both chemical and energy costs required for water production and distribution. Reducing the number of UDF runs can also provide a cost savings with the upcoming regulation concerning dechlorination. Optimizing the amount of water discharged and therefore requiring dechlorination, a potentially expensive process, will be a great benefit.

Table 3 Proposed schedule of unidirectional flushing runs

Year	2002	2003	2004	2005	2006	Five-Year Total
Number of UDF runs	1,355	1,827	2,508	1,576	2,079	9,346

Table 4 Summary of study results

	2002 Actual	2002–2006 Proposed
Number of UDF runs	1,355	9,346
Cost savings (in Canadian dollars)	$150,000	$329,183

Time Efficiencies

Preparation for the UDF Program begins for the next season as soon as the flushing for that year is completed. The field records are input into the database, and the generation of the next year's design forms and maps begins back in the office and continues up to the moment that the next season is ready for flushing to start again. With fewer UDF runs to prepare for, much time will become available for other applications in the UDF program. Office activities, such as redesign of existing areas to incorporate the changing pipe material ratios and application of the hydraulic model to confirm water flows and velocities, will enable ongoing improvements in the quality of flushing and therefore the quality of the water distribution system.

CONCLUSIONS

In 2002, EPCOR Water Services Inc. took a step back and questioned their existing Unidirectional Flushing Maintenance Program. With efficient flushing, large cost savings can be realized through reduced manpower, energy consumption, and water use. The 2002 UDF Frequency Optimization Study was a success because information organization enabled an informed and confident decision. A four-month study was conducted that compiled seven years of UDF records and analyzed the available data based on intelligent influential factors. The decision to extend flushing frequencies up to four years resulted from an invested $8,000 study; study recommendations resulted in savings of approximately $150,000 Canadian for the 2002 flushing season.

The ability to optimize UDF flushing frequencies depends on the quantity and quality of past flushing data. EWSI felt confident in revising their UDF frequencies after analyzing seven years of data collected through Edmonton's citywide biennial Unidirectional Flushing Program. Although no industry-wide standard currently exists for the frequency of flushing, recommended annual or biennial flushing is a known rule of thumb. The condition of the infrastructure and the water quality in the distribution system will affect the allowed time between flushings. With expertise in UDF, EWSI prepares UDF programs for a number of client communities. In addition to providing experience and knowledge for design and data maintenance, EWSI can now provide the service of flushing frequency optimization, even for first-time implementation of this effective pipe-cleaning technique. EWSI recommends regular, frequent water quality monitoring and data record maintenance so that flushing frequency can be analyzed as the UDF Program progresses.

The Unidirectional Flushing Maintenance Program is essential for EPCOR Water Services Inc. to provide high water quality and service to its customers. The program is continually critiqued for potential areas of improvement and efficiencies, including the optimization of flushing frequencies.

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2. Deagle, G. (2003). Optimising Flushing Programs of Water Distribution Systems. *Master in Engineering Non-thesis Project*, University of Alberta, Edmonton, AB, 97p.

APPENDIX A: AREA INFORMATION FREQUENCY SHEET

Table A1 Basis statistics

Year	Number of Runs	Avg. Length of Run (*m*)	Number of Valves	Number of Hydrants	Avg. Length of Hydrant Lead (*m*)
1995	123	425	n/a	n/a	n/a
1997	123	425	n/a	n/a	n/a
1999	122	247	n/a	n/a	n/a
2001	118	433	482	208	4.84

Table A2 Length (*m*) of pipe by material

Year	Unlined CI	PVC	AC	CI Epoxy-Lined	Others	Total
2001	42,322	6,333	1,650	0	602	51,134

Table A3 Flushing history

Year	Number of Runs Flushed	Total Flush Time (*hr*)	Total Consumption (*ML*)
1995	123	61.87	11.403
1997	115	51.35	12.668
1999	115	43.03	8.708
2001	112	51.25	8.591

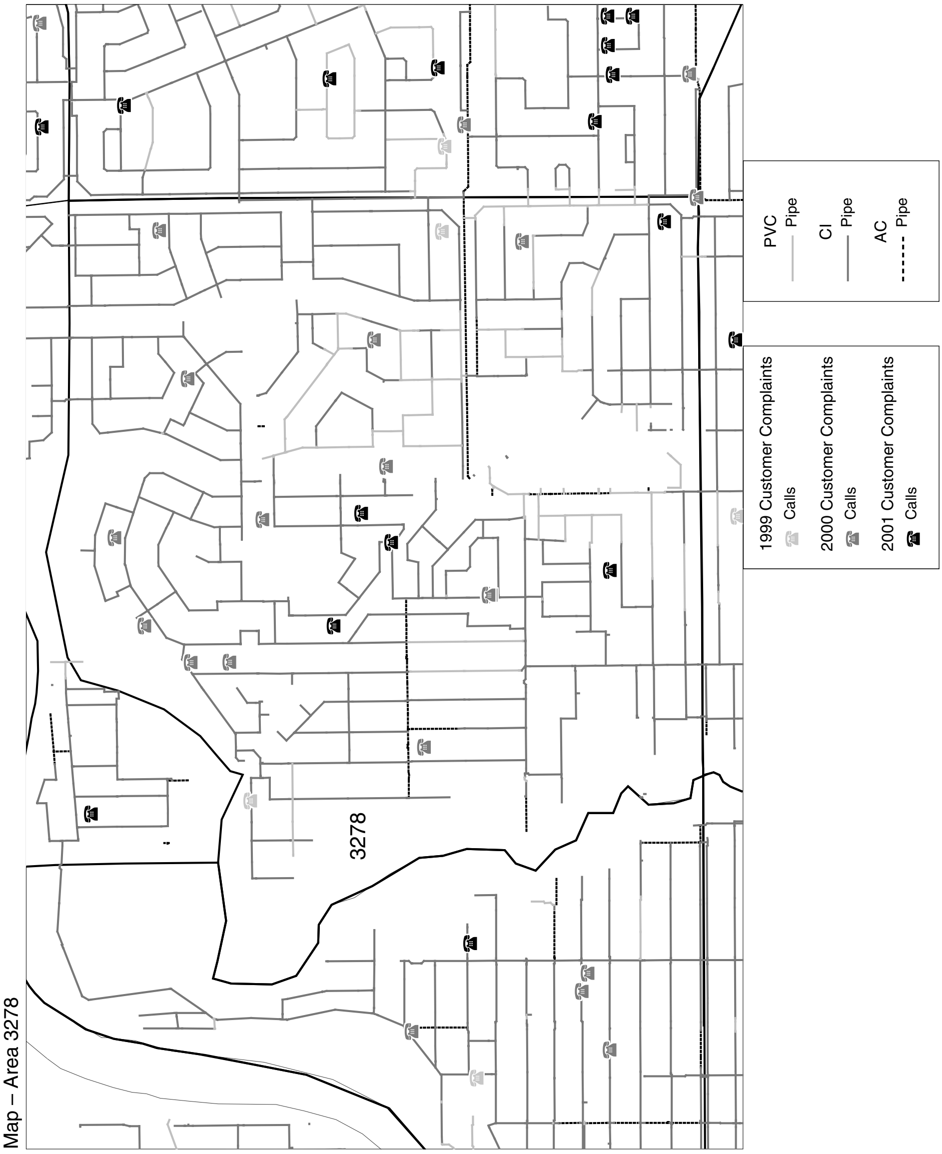
Map – Area 3278
3278
1999 Customer Complaints
Calls
2000 Customer Complaints
Calls
2001 Customer Complaints
Calls
PVC
Pipe
CI
Pipe
AC
Pipe

Table A4 Water quality complaints

Year	Turbidity/ Color	Odor and/or Taste	Hardness	White Particles	Illness	Other	Total
1999	0	1	0	0	0	1	2
2000	12	1	2	1	0	0	16
2001	5	1	0	0	0	0	6

NOTE: Hydroscope Program in 2000 and 2001 caused higher turbidity and color complaints.

Table A5 Preflushing water quality sampling averages, by area

Year	Residual Cl_2 (*mg/L*)	HPC (*CFU/mL*)	Turbidity (*NTU*)
2001	1.97	1.08	0.07

Table A6 During flush averages

Year	Start Turbidity (*NTU*)	End Turbidity (*NTU*)	Water Used (*L*)/run	Flushing Time (*min*)/run
1995	2.76*	1.45	92,707	30.18
1997	27.00	1.36	110,157	26.79
1999	74.59	1.04	75,722	22.45
2001	67.90	1.21	76,705	27.46

* Only approximately half of the runs had start and end turbidity data.

Table A7 Postflushing water quality sampling averages, by area

Year	Residual Cl_2 (*mg/L*)	HPC (*CFU/mL*)	Turbidity (*NTU*)
2001	1.81	0.71	0.12

Table A8 Pre- and postflushing water quality comparison, by run

	Number of Runs With > 50% Improvement		
Year	Residual Cl_2 (*mg/L*)	HPC (*CFU/mL*)	Turbidity (*NTU*)
2001	0 out of 0	2 out of 18	1 out of 13

Table A9 Assessment of the effectiveness of the most recent flushing exercise

	Actual		Trigger	
Parameter	Value	Percentile	Value	Percentile
Percent of total pipe length that is cast iron	83%	94%	65%	>75%
Average length of hydrant leads	4.8 m	91%	3.3 m	>75%
Average flushing time/run	27 min	90%	24 min	>75%
Field turbidity at start of flush	68 NTU	94%	43 NTU	>75%
Number of total water quality complaints	6	67%	3–4	>50%

Recommended Flushing Frequency: 1-year cycle
Next Flushing Year: 2002

CHAPTER · 47

Automatic Flushing: A Solution for Enhancement of Distribution System Water Quality

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SUMMARY

The integration of distribution system water quality management practices appears to be among the future regulatory requirements that utilities can anticipate facing in the near future. Historically, water utilities have incorporated programs for distribution system flushing as a means of restoring water quality, particularly within dead ends or extremities of their systems. These practices usually require coordination with distribution system and laboratory personnel for the flushing of sections of the system that may be experiencing less than desirable water quality variations. Needless to say, the time and coordination effort required among these personnel can be financially burdensome depending on the flushing frequency and water quality conditions experienced.

As part of an initiative to enhance water quality within the distribution system, the City of North Miami Beach Public Services Department has incorporated several automated water flushing devices to flush certain areas of the distribution system on a scheduled basis. The automatic flushing system uses a fraction of the water otherwise required by a manual flush, but more importantly it does not require personnel for its operation. Although flushes occur periodically, they are not voluminous and thus have insignificant impact on area drainage. However, more important are the water quality benefits achieved when conducting periodic flushes on a scheduled basis. Since the operational implementation of these devices, chlorine residuals, pH, and total trihalomethanes have improved in problematic areas of the distribution system in comparison with previous years. This chapter will provide an analysis of quantifiable benefits achieved by the city in overall water

quality as well as operational cost savings associated with the use of an automated flushing device.

INTRODUCTION

The City of North Miami Beach Public Services Department provides water services to approximately 200,000 people within the city and adjacent area of Miami–Dade County, Fla., through about 500 mi of pipelines in the distribution system. The city owns and operates the Norwood Oeffler Water Treatment Plant, which produces about 15 million gallons per day (mgd) of lime-softened water. The city purchases water from Miami–Dade Water and Sewer Department (MD–WASD) to fulfill the system's water demand. Both Norwood Water Treatment Plant and MD–WASD water are disinfected by chloramination.

To maintain a desirable chlorine residual level in the distribution system, it is important to prevent biological activities within the distribution system and to provide adequate circulation of water in the system. Historically, the Public Services Department has relied on programs such as annual distribution system flushing, annual free chlorine flushing, and point flushing to maintain chlorine at a desirable residual level. These flushing methods have historically provided water quality improvement. In addition, the system monitors pH, alkalinity, phosphate, and temperature for the Lead and Copper Rule. When pH or other parameters are below the required level, flushing is mandatory. In 2000, the city of North Miami Beach started to search for innovative ways to improve the chlorine residuals and other parameters in the distribution systems, especially for system extremities and dead ends, not only to improve and maintain constant water quality throughout the distribution system, but also to reduce excessive overtime labor costs.

AUTOMATIC FLUSHING SYSTEM PILOT STUDY

Two automatic flushing devices, called Hydro-Guard, manufactured by Environmental Enhancement & Technologies USA, Inc., were purchased by the city to evaluate performance with respect to chlorine residual improvement. Two locations were carefully selected for the installation of the two automatic flushing devices. Both locations had low chlorine residuals and pH historically, and both locations have optimum drainage areas. One location has a catch basin about 15 ft away downgrade; the other location is a well-sodded area. A meter and a check valve were installed with each automatic flushing device by city staff. Chlorine residual decay was tested for both devices to determine the flushing schedules. As a result, the devices were set up for flushing 10 min on Mondays and Thursdays, respectively.

The Hydro-Guard Standard Unit consists mainly of a solenoid-operated valve; a 14-day, 16-event battery operated programming pod; an atmosphere discharge with an air gap to prevent backflow; and a sample port. A handheld programmer can be connected to program the Hydro-Guard so that the controller opens and closes the solenoid-operated valve at any preset days, times, and for the corrected duration. The programmer can be disconnected after programming. A typical installation of the automatic flushing device is shown in Figure 1.

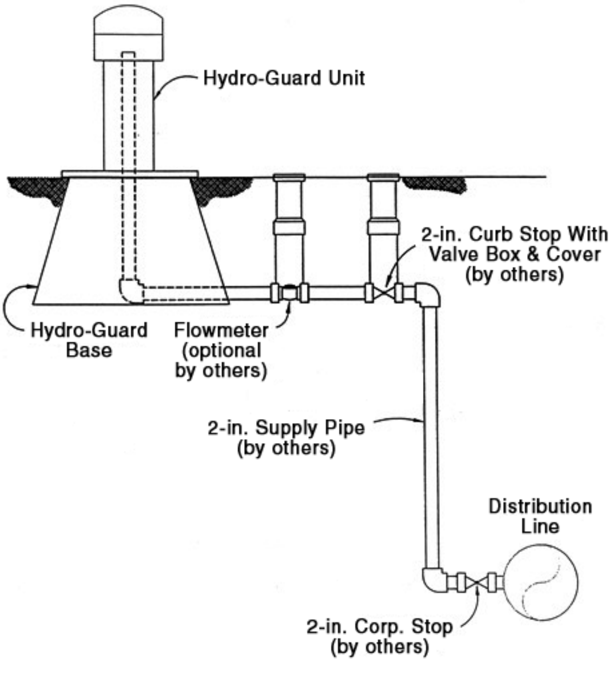


Figure 1 Automatic flushing device installation

EVALUATION OF AUTOMATIC FLUSHING SYSTEM

Studies have shown that compared with manual flushing, automatic flushing can keep constant chlorine residuals because it can be programmed to flush more frequently. The schematic in Figure 2 demonstrates chlorine residual decay when manual or automatic flushing is used.

To evaluate the long-term results of the automatic flushing devices in the area, the chlorine residual data from 1998 to 2002 were analyzed at five locations closest to the two automatic flushing locations. These five locations are shown in Table 1. The sixth location was chosen to

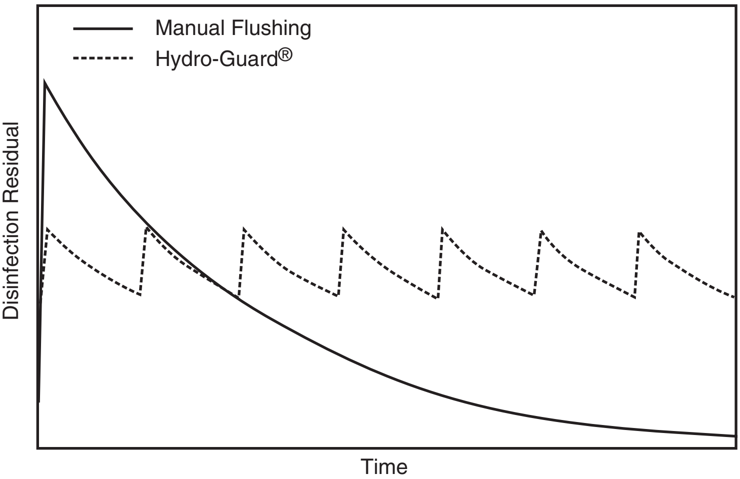


Figure 2 Chlorine residual decay after manual flushing and automatic flushing

Table 1 Selected locations closest to the automatic flushing devices

Location	Close to
101 121	Hydro-Guard #1
104 123 120	Hydro-Guard #2
102	In the area

demonstrate the effect of the hydro flushers on the area. The location is not immediately adjacent to the flushers but had historically low chlorine residuals.

The city routinely monitors the chlorine residuals in 134 locations throughout the distribution system; therefore, data are available for comparison between before and after the automatic flushing devices were installed. The five locations selected are monthly monitoring locations that are close to the installation of the automatic flushing devices.

Chlorine Residual Improvements

Figures 3 through 8 demonstrate the improvement of the chlorine residuals in the area. Locations 101 and 121 had significantly higher chlorine residuals in 2002, after the automatic flushing device was in operation (Figures 3 and 4). Historically, the chlorine residuals had been low, sometimes as low as 0.1 mg/L. Since the automatic flushing device was installed, all the data were over 1.0 mg/L except two at 0.5 mg/L. Location 120 also showed significant improvement; the chlorine residuals were over 2.5 mg/L since the installation (Figure 5).

Locations 104 and 123 also demonstrated improvement. However, the effect was not so dramatic because chlorine residual results were not

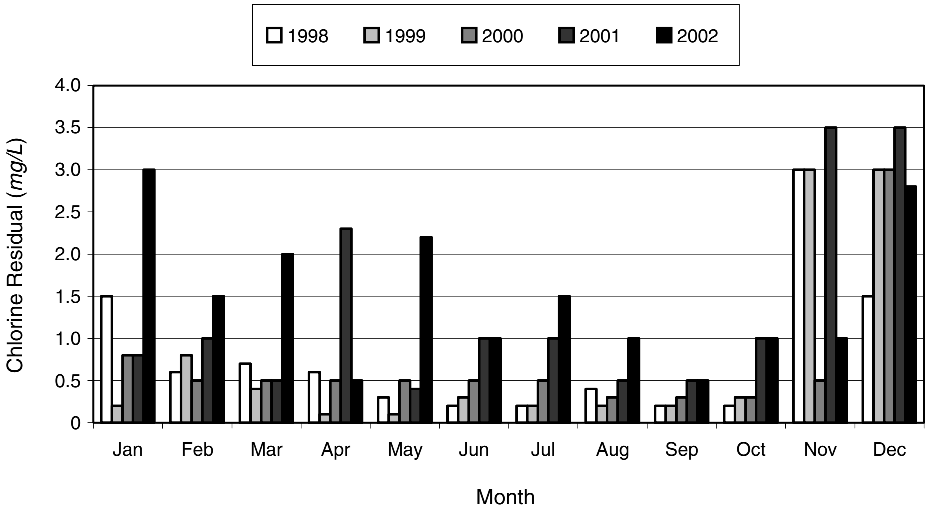


Figure 3 Chlorine residuals from 1998 to 2002 at Location 101

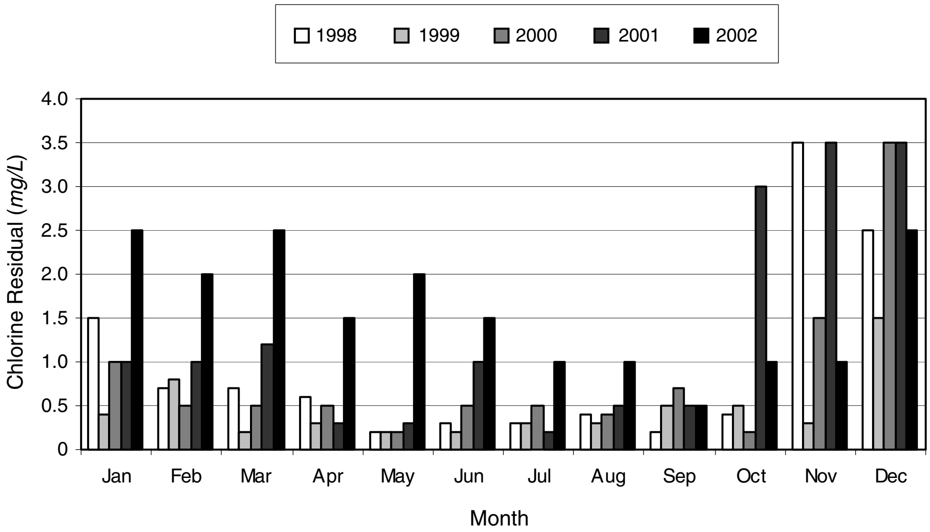


Figure 4 Chlorine residuals from 1998 to 2002 at Location 121

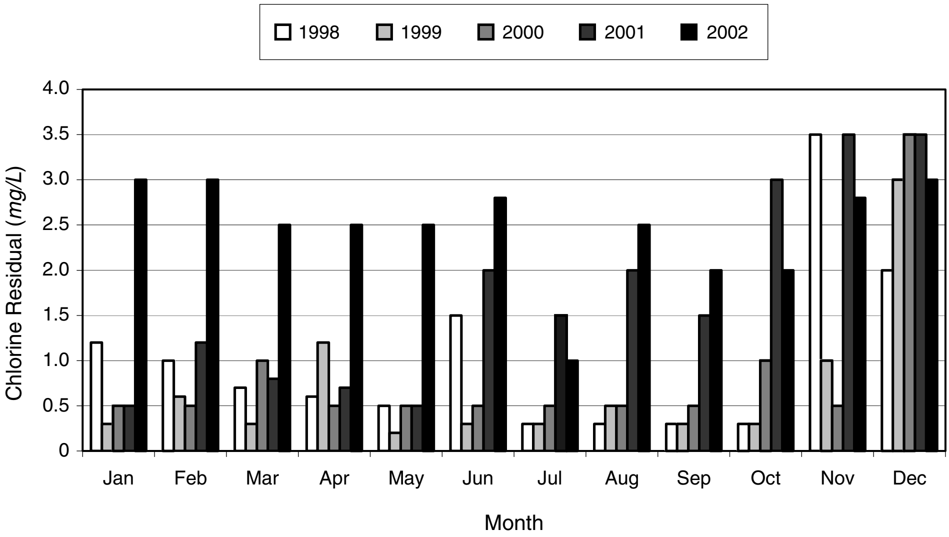


Figure 5 Chlorine residuals from 1998 to 2002 at Location 120

significantly low even before the automatic flushing device was installed (Figures 6 and 7).

It is also interesting to see the effect of the hydro flushers on the whole area. There was a perfect location to check the effect. Location 102 had historically been low in chlorine residuals, and it is not immediately adjacent to the two hydro flushers. Hydro flusher #1 is close to one mile away, and hydro #2 is more than one mile away. The data clearly demonstrated improvement in chlorine residuals in this location, as shown in Figure 8.

It should be noted that chlorine residuals in October through December for all the locations were high. The reason is that every year at the end of October or in early November, the city of North Miami Beach conducts annual free chlorine flushing. This free chlorine flushing helped boost chlorine residuals significantly throughout the distribution system. Also, some of the unusually high numbers for chlorine residuals could be the result of manual flushing. Data were not available for confirmation.

It should also be noted that either before or after the installation of the hydro flushers, there was a trend that the chlorine residuals declined after April or so. This is a cumulative effect after the free chlorine flush. The same flushing schedule was kept throughout the year for the flushers,

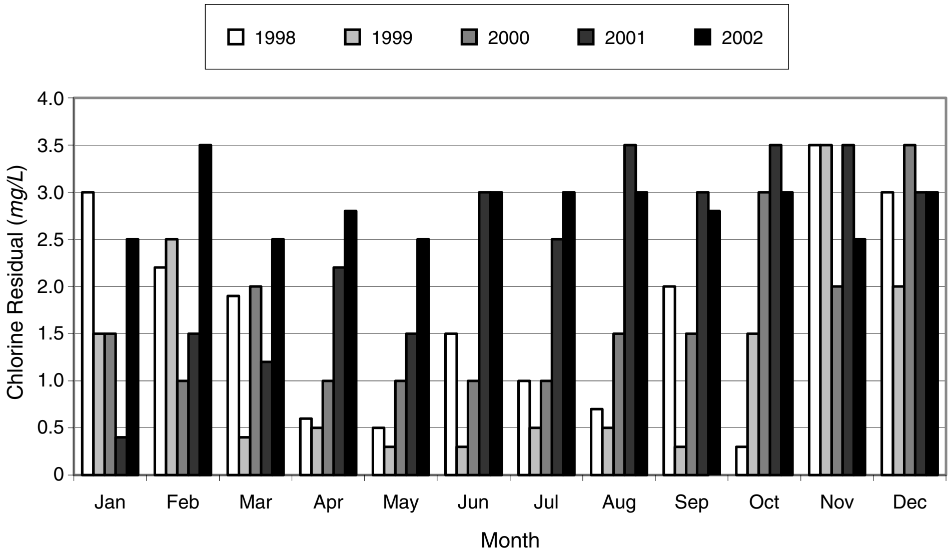


Figure 6 Chlorine residuals from 1998 to 2002 at Location 104

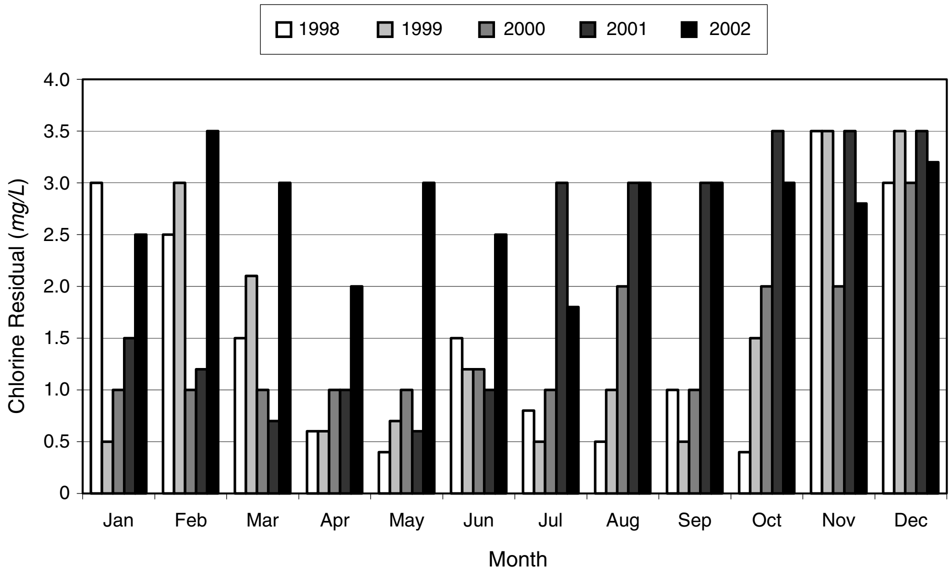


Figure 7 Chlorine residuals from 1998 to 2002 at Location 123

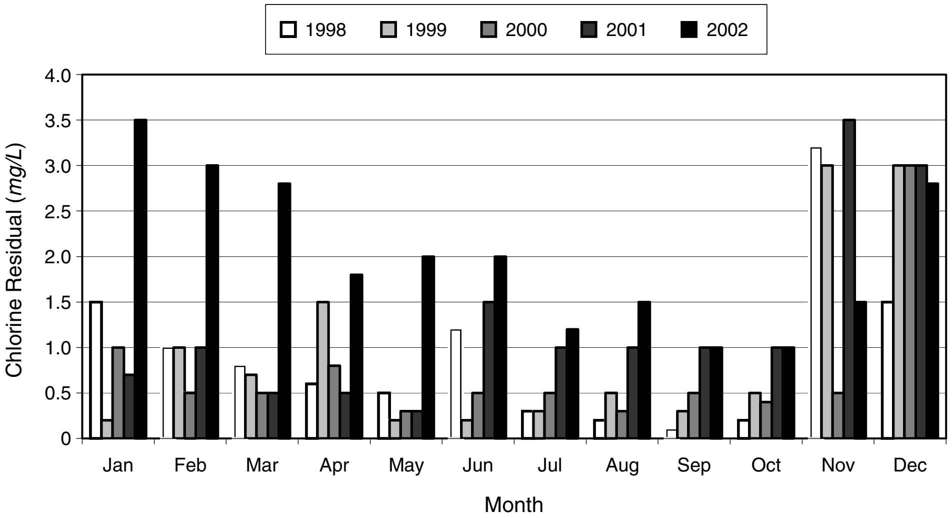


Figure 8 Chlorine residuals from 1998 to 2002 at Location 102

and thus the decline was also observed in 2002. However, that was a lesson. The flushing schedule can be adjusted based on this information.

Table 2 shows the yearly average chlorine residuals for the five locations from 1998 to 2002. Significant chlorine residual improvements were observed. The relatively high chlorine residuals in 2001 were the result of installation testing of the automatic flushing devices. When the flushing device was first installed, some problems were experienced. The devices worked sometimes but not other times. The devices have been in full operation since January 2002.

pH, Total Trihalomethane (THM), and Turbidity Improvements

As required by the Florida Department of Health based on the Lead and Copper Rule, the city monitors pH in the distribution system. Based on the requirement, the distribution system pH cannot be lower than 8.5. Whenever distribution system pH is lower than 8.5, the city is required to flush the area.

Although it was not well documented, there had been some manual flushing due to low pH in the area. Since the installation of the automatic flushing device, there has been no pH problem in the area. Because of frequent flushing, there is less chance for sediments to deposit in the distribution system, especially for far ends and dead ends. Therefore, customer complaints are significantly reduced.

One of the locations, 121, is also a monitoring location for THM. The total THM results for this location from 1999 to 2002 are listed in Table 3. The results demonstrate decreasing THM since the automatic flushing device was installed (2002).

Table 2 Yearly chlorine residual averages

Location	101	121	104	123	120	102
1998	0.783	0.942	1.683	1.558	1.017	0.925
1999	0.733	0.458	1.15	1.55	0.692	0.950
2000	0.683	0.833	1.667	1.433	0.833	0.733
2001	1.333	1.333	2.4	2.125	1.725	1.250
2002	1.500	1.583	2.842	2.775	2.467	2.008

Table 3 Average THM from 1999 to 2002 for location 121

Year	1999	2000	2001	2002
THM (ppb)	12.28	24.30	22.05	11.57

Other Benefits

In addition to water quality improvement, automatic flushing has the following benefits over manual flushing.

- The automatic device flushes at a designated time and no labor is required during the flushing, while manual flushing requires several workers, usually working at night, leading to increased costs for overtime.
- By flushing small amounts of water more frequently, water savings can be achieved. The study showed that savings can be as high as 80%.
- The amount of water discharged is significantly less than with manual flushing. Because the location was carefully selected, no local flooding was observed.
- Reduced customer complaints due to improved water quality, especially turbidity.

COST SAVINGS

Table 4 shows the cost analysis for automatic flushing compared with manual flushing. Automatic flushing has significant cost savings over manual flushing when manual flushing is needed at least twice a year. The more manual flushing time, the more cost savings for automatic flushing. The following assumptions are used for the analysis:

- A standard unit of the automatic flushing device currently costs $2,245.
- Supplies such as meters and check valves; in-house installation; and startup cost $1,500.
- A half-hour quarterly maintenance is required for the automatic flushing device.

Table 4 Cost analysis for automatic flushing and manual flushing

	Capital Invest.	Uniform Amount per Year for Capital Invest.	Yearly Maintenance Labor	Yearly Maintenance Supply	Yearly Total
Automatic flushing	$3,745	$215	$32	$50	$297
Manual flushing once	0	0	$312	$30	$342
Manual flushing twice	0	0	$624	$60	$684
Manual flushing three times	0	0	$936	$90	$1,026
Manual flushing four times	0	0	$1,248	$120	$1,368

- Supplies for the maintenance of the automatic flushing device cost $50 per year.
- The service life of the automatic flushing device is 25 years.
- One superintendent and two maintenance workers are needed for manual flushing.
- Employees are paid at least four hours' overtime for each manual flushing.
- The overtime labor fee for the superintendent is $30 per hour; the overtime labor fee for the maintenance workers is $24 per hour.
- The inflation rate is assumed to be 3%.

The uniform amount per year for capital investment is calculated based on the following equation:

$$A = \frac{i(1+i)^n}{(1+i)^n - 1} \times P$$

Where:

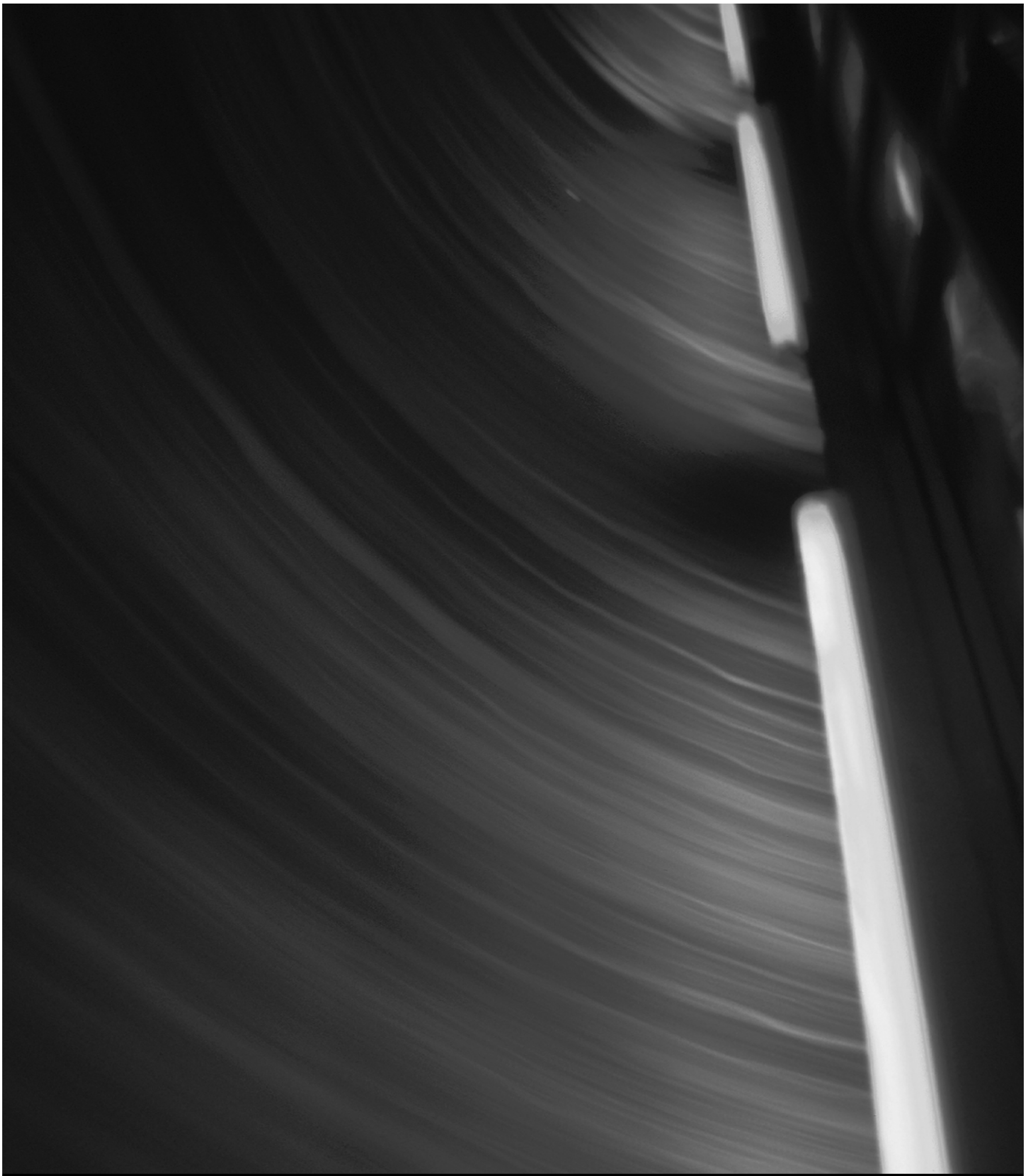
A = Uniform amount per year for capital investment
P = Present valve for capital investment
i = Interest rate, 3%
n = Return period, 25 years

CONCLUSIONS

This chapter presents a case study of using automatic flushers to boost chlorine residuals for dead-end or far-reach areas of the distribution system. Chlorine residual data after the installation of the automatic flusher were compared with historical data in six distribution points adjacent to the flusher. A cost analysis was also conducted to compare the economics of using automatic flushers with manual flushing. The results indicated that by flushing twice a week with limited flow, the chlorine residuals improved significantly. In addition, pH level was also increased, eliminating the manual flushing program that was needed for lead and copper water quality parameter monitoring. Automatic flushing has significant advantages over manual flushing, namely improved and constant water quality, including chlorine residual, pH, THM, turbidity, cost savings, water savings, labor savings, and reduced customer complaints.

ACKNOWLEDGMENTS

The authors thank Gary Garafalo, who was in charge of the installation of the automatic flushing devices, and Environmental Enhancement & Technologies USA, Inc., for providing useful information about the device.



Part 9

Water Quality Computer Modeling (Computer-Aided Network Analysis)

CHAPTER · 48

Managing Water Distribution Systems for Potential Intrusions Using Integrated Modeling and GIS

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SUMMARY

Water utility personnel who manage drinking water distribution systems face many challenges. They balance operation and maintenance tasks under cost, reliability, and regulatory constraints. Their tasks are complicated by the relative age of the drinking water distribution systems. Aging systems are of concern due to unaccounted-for water, increased incidence of breakage, and the potential for water quality degradation owing to intrusion events. A unified approach is proposed that addresses the susceptibility of drinking water distribution systems to intrusion events. This approach employs infrastructure information, a hydraulic model, and demographic data. These data are managed within a geographic information system (GIS). The framework systematically identifies pipe segments susceptible to microbial intrusions and prioritizes these segments for attention based on knowledge about nearby sensitive populations. The results of the decision framework may support utility capital improvement plans and infrastructure maintenance, and can be used as a basis for sampling regulatory schemes.

INTRODUCTION

Treatment of drinking water is often referred to as a multiple-barrier approach. The barriers may include source water protection, treatment technologies, primary disinfection/oxidation, and finally a disinfectant residual in the distribution system. These measures have as their primary goal the protection of public health and generally result in a safe water supply with relatively few reported waterborne disease outbreaks. The final barrier in the drinking water supply, the distribution system, is coming under increasing scrutiny. Of waterborne disease outbreaks occurring between 1971 and 1994, 30% of 272 outbreaks (in community systems) reviewed have been attributed to problems with the distribution system (Shaw and Regli, 1999). A recent roundtable on disinfectant residuals noted that contamination may occur in the distribution system on a daily or even hourly basis (AWWA, 1999). These contamination events may result from many factors, including pipe age, soil stability, joint leakage, low or negative pressure, uncontrolled cross-connections, and proximity to pathogen sources. It is generally accepted that the microbiological quality of treated drinking water as it leaves the treatment plant is the highest it will ever be (AWWA, 1999).

BACKGROUND: MULTIPLE OBJECTIVES AND CONSTRAINTS

A primary objective in the design and construction of drinking water distribution systems is reliability—providing the consumer with the flow needed at a desirable pressure at all times. This objective is met by building a larger system (in terms of pipe size) with more storage. Larger systems provide a safety factor and allow demands to be met during peak periods or in the case of fire flows. However, system redundancies essential for reliability are counter to the needs for delivery of water that is relatively unchanged from when it left the treatment plant. Larger, more redundant systems increase the water age, which may in turn result in excessive loss of chlorine residual and excessive production of disinfection by-products. An optimal combination of system operation and maintenance involves balancing hydraulic reliability and the risk of impaired water quality (Grayman et al., 2000).

System Age

Underlying water quality concerns and reliability are the overall age and condition of distribution systems in the United States. A recent survey of water utilities in the US showed that the average age of the oldest section of pipe is 79 years, with pipes over 100 years old reported as still in service. The same survey also found the average amount of unaccounted-for water to be between 15% and 20% of the total volume put into the distribution system (Haas, 1999). The 1997 Infrastructure Needs Survey (USEPA, 1997) estimated the cost to upgrade transmission and distribution systems in the US at $77.2 billion. This was based on a 20-year planning horizon to meet water quality standards. Storage needs were an additional $12.1 billion. These costs represent 56% and 9% respectively of the total drinking water infrastructure cost estimates given in the 1997 report (including distribution, storage, treatment, source water protection, etc.). Thus utilities continually search for ways to prioritize operations, maintenance, rehabilitation, and system monitoring (sampling) decisions. Many utilities aggregate maintenance records, consumer complaints, knowledge of leakage, flushing data, and location of existing facilities in an effort to make educated operational decisions. Utilities are also relying more on hydraulic and water quality models to investigate distribution system attributes such as low- or high-pressure regions, disinfectant residual, and water age.

Regulatory Burden

System monitoring of certain constituents is required for regulatory compliance under sections of the Safe Drinking Water Act (SDWA). One regulation in particular, the Total Coliform Rule (TCR), requires a certain number of samples to be collected based on the number of people served. The TCR, however, gives little guidance on where in the distribution system the samples should be collected or how many monitoring locations should be employed. Other regulations influencing distribution system management include rules for chlorine residuals, monitoring of disinfection by-products, and monitoring of lead and copper (Grayman et al., 2000). Some water professionals have suggested that a more comprehensive management approach that employs quality or best management practices would serve the public better than regulations that require extensive sampling and are possibly of little value (Allen et al., 2000).

Populations Served

Under the 1996 Safe Drinking Water Act Amendments, Congress asked the United States Environmental Protection Agency (USEPA) to continue to identify groups that may be at greater risk than the general populace due to contaminants in drinking water. These sensitive subpopulations are defined by USEPA as those groups or individuals who respond biologically to lower levels of exposure to a drinking water contaminant or whose response has more serious health consequences. Subpopulations such as infants, children, the elderly, and those with compromised immune systems are groups currently under review. Table 1 shows how these groups are proportioned in the United States (USEPA, 2000).

Table 1 Sensitive subpopulations in the United States (after USEPA, 2000)

Subpopulation	Number of Individuals	Percentage of Population
Pregnant women	6,240,000	2.4%
Infants and children (<10 years)	38,704,000	14.1%
Elderly (>65 years)	34,817,000	12.6%
Diabetic	15,700,000	5.8%
Liver impairments	595,000	0.2%
Immuno-compromised	400,500	0.2%

The USEPA has suggested that future regulations could address the aging population and their need for special health considerations (USEPA, 1999). A comprehensive distribution system management plan has been suggested that could include consideration of the locations of sensitive populations (Antoun et al., 1999).

OBJECTIVES

To address the problems posed above, research is conducted that considers both distribution system information and the populations served to support management decisions related to possible contaminant intrusions in the distribution system. Population information may provide the final decision basis when distribution system information alone will not yield clear direction. The objectives are as follows:

- Formulate a decision support framework that (1) addresses the potential for microbial contaminant intrusions based on distribution system information, and (2) prioritizes management (e.g., repair, operational adjustments, or monitoring) of susceptible pipe segments based on population information.
- Develop the framework considering data typically available to a water utility, the tools of database management, geographic information systems (GIS), and a distribution system hydraulic/water quality model.
- Apply the framework to an actual distribution system.

The proposed framework can be applied at any utility. It finds pipe segments in the distribution system that are most susceptible to an intrusion event. Other appurtenances, such as tanks, are not addressed in this work. The framework does not address a gross contamination event such as treatment plant upset or placing a pipe in service without proper disinfection procedures.

EXPERIMENTAL METHOD

Three steps summarize the experimental method:

1. Contaminant intrusion and demographic factor identification
2. Decision framework development
3. Application of framework to an actual utility

Contaminant Intrusion and Demographic Factor Identification

Conceptually, the mechanisms or events leading to an intrusion in a pipe can be split into two groups. The first mechanism is a subsurface intrusion somewhere along the pipe wall. For this to occur the pressure inside the pipe needs to be very low or lower than the external pressure (if the pipe were submerged). Also, a compromised location along the pipe wall or at a joint is required for contaminant entry (i.e., crack, offset joint, etc.) and pathogens must be present to yield a pathogenic intrusion event. The second mode of intrusion is at a location where the potable water supply is connected to a customer's internal water or process supply. Obviously, these connections occur throughout the distribution system, and certain types of connections (namely industries) are controlled through a utility's cross-connection control program. In the event that internal process pressures exceed the pressure on the potable side of the system, backflow is possible. If appropriate controls are not in place, backflow can lead to contamination of the potable water supply.

The first mode of intrusion (subsurface along the pipe wall) is conceivable during a depressurization event, such as extreme demand coupled with fire flow, a pipe break, or hydraulic transient. LeChevallier (1999), citing an ongoing AwwaRF project on transients, reports these kinds of intrusions during negative pressures generated from both a pump failure and a main break. He notes that during the main break the negative pressure may occur at points in the distribution system some distance from the break. Relative to the second mode of intrusion (cross-connection), Trussel (1999) cites Geldrich (1996) and notes that inadequate pressure and back-siphonage (cross-connection) are, by far, the most common sources of distribution system contamination worldwide.

Combining a compromised pipe or problematic cross-connection with a potential receptor, in this case the sensitive population, gives rise to a conceptual model of an exposure scenario. Conceptual models of this type have been used to estimate the risk to human health due to some type of exposure, either chemical or biological (ILSI Risk Science Institute, 1996). The AwwaRF guidance manual for water main evaluation and rehabilitation (O'Day et al., 1986) provides decision logic that suggests that utilities consider the risk to critical facilities or the risk of property damage when evaluating pipes to replace. Others have used this approach—the likelihood of breaking combined with the potential impact—to manage buried infrastructure (Hadzilacos et al., 2000; Fenner and Sweeting, 1999; Griffis and Ivey, 1998). The framework herein is an extension of this approach. The potential for intrusion is coupled with the potential impact to downstream sensitive populations. Given the two modes of intrusion and the sensitive receptors, specific data sources are identified that a utility may want to access to begin implementing the framework discussed in the following sections. Table 2 gives examples of the type of data needed to implement the framework.

Intrusion variables as data layers

Successful implementation of the decision framework requires that variables shown in Table 2 be available to the utility in some sort of digital format. Digital examples include database files, spreadsheets, aerial photography, and digital maps (e.g., CAD files). The digital variables then become layers within the proposed distribution system decision framework. A geographic information system (GIS) is the logical repository for the data layers. These data layers can be considered as map layers when spatially tied to each other using the same map projection and coordinate system. Each entity in the database has both spatial reference data and attribute data. For example, a pipe has a northing and easting to locate it on the map and an associated attribute table with information such as age and diameter. Direct implementation of the framework is essentially a series of successive operations or queries that translate available "primary" data layers into "derived" data layers, which will allow one to identify areas in the distribution system that are susceptible to intrusion events. An example of such a translation or reclassification of data layers is assigning a plasticity rating to a soil type where the plasticity rating may be a good indicator of structural integrity and thus potential leakage.

Table 2 Example data needs assessment for distribution system

Variable	How Variables Impact System	Data Source	Comments
System pressure	Low or negative pressure is necessary for intrusion into a pressurized system	Calibrated hydraulic model, pressure data	Pressures below 20 psi are undesirable; many utilities have target minimums of about 30 or 35 psi
Pipe attributes (age/vintage, material, size)	May be an indicator of structural integrity and possible leakage	Utility asset records and historical repair data	Historical construction practices and subsequent repair observations show some pipe vintages have a high incidence of leakage and breakage
Bedding conditions (slope, soil type)	May be an indicator of structural integrity and possible leakage	Soil map, contour map, digital elevation model	Slopes can be evaluated for stability; soils can be ranked for corrosiveness or shrink-and-swell potential
Proximity to pathogens (sewers, etc.)	Sewer lines are a source of pathogenic organisms and may be buried near potable water lines	Utility maps	Standard construction specifications typically require a minimum offset or guidance when crossings are required
Chlorine residual	Provide barrier against undesirable intrusions	Calibrated water quality model	Rapid unexplained decrease may suggest existing intrusion; provides barrier to intrusion event

Table continued next page.

Table 2 Example data needs assessment for distribution system *(continued)*

Variable	How Variables Impact System	Data Source	Comments
Children and infants: day care centers, preschools, elementary schools	Young or very young often cited as a sensitive population	Health department, phone book	Mobility of these institutions (e.g., children are not always there) may need to be considered in analysis
Elderly: assisted living homes, retirement homes, nursing homes	Elderly often cited as a sensitive population	Health department, phone book	Relative stability of these institutions may simplify decision framework for this group of people
Hospitals	Already afflicted or compromised often cited as a sensitive population	Phone book	Hospitals tend to be a large population center of both immuno-compromised and healthy individuals

Decision Framework

The decision framework is a method to synthesize all of the contributing variables into coherent information. With the variables together in a coherent format, utilities can establish decision criteria based on knowledge about their systems to target areas of the system for attention. The use of potentially impacted populations in making these operational decisions is a novel approach and is included to provide an additional decision point for utility managers when information about the system only is not providing a clear basis for operation or maintenance decisions. Figure 1 shows the key components of the decision framework. The figure is discussed in detail below.

Pressure analysis

Low or negative pressures make pipes vulnerable to inflow from cross-connections or at locations where the structural integrity is compromised. For intrusion along a pipe the external fluid pressure must exceed the internal fluid pressure. Similar for backflow at a cross-connection, the back-pressure must exceed the internal pipe pressure. The pressure analysis can be implemented either using a calibrated hydraulic model of the system or possibly with pressure data collected at locations throughout the system; a calibrated hydraulic model is recommended and was used for this work. EPANET was chosen due to its availability and user access to the source code. Regardless of the hydraulic model used by the utility, the output needed is essentially the same; a time history of pressures at all nodes being considered during a specified period (e.g., 24 hr) is required. From this point the data are queried for such data as minimum pressure or the amount of time a node experiences pressure lower than some value. Possible low-pressure regions are identified spatially in the GIS. Actual decision points or query criteria are determined by the utility implementing the framework.

Leak/structural integrity analysis

Even if low pressure is occurring, a lack of structural integrity in the form of a leak or cross-connection must exist for an intrusion to occur. The simplest and most conservative approach for investigating the structural integrity of the system is to assume that all pipe segments, valves, joints, and other appurtenances are leaking or have cross-connections. Under this assumption, only a pressure analysis would have to be conducted. Pipes failing a utility's pressure criteria would be those susceptible to intrusion events. A more rigorous approach is to consider data available

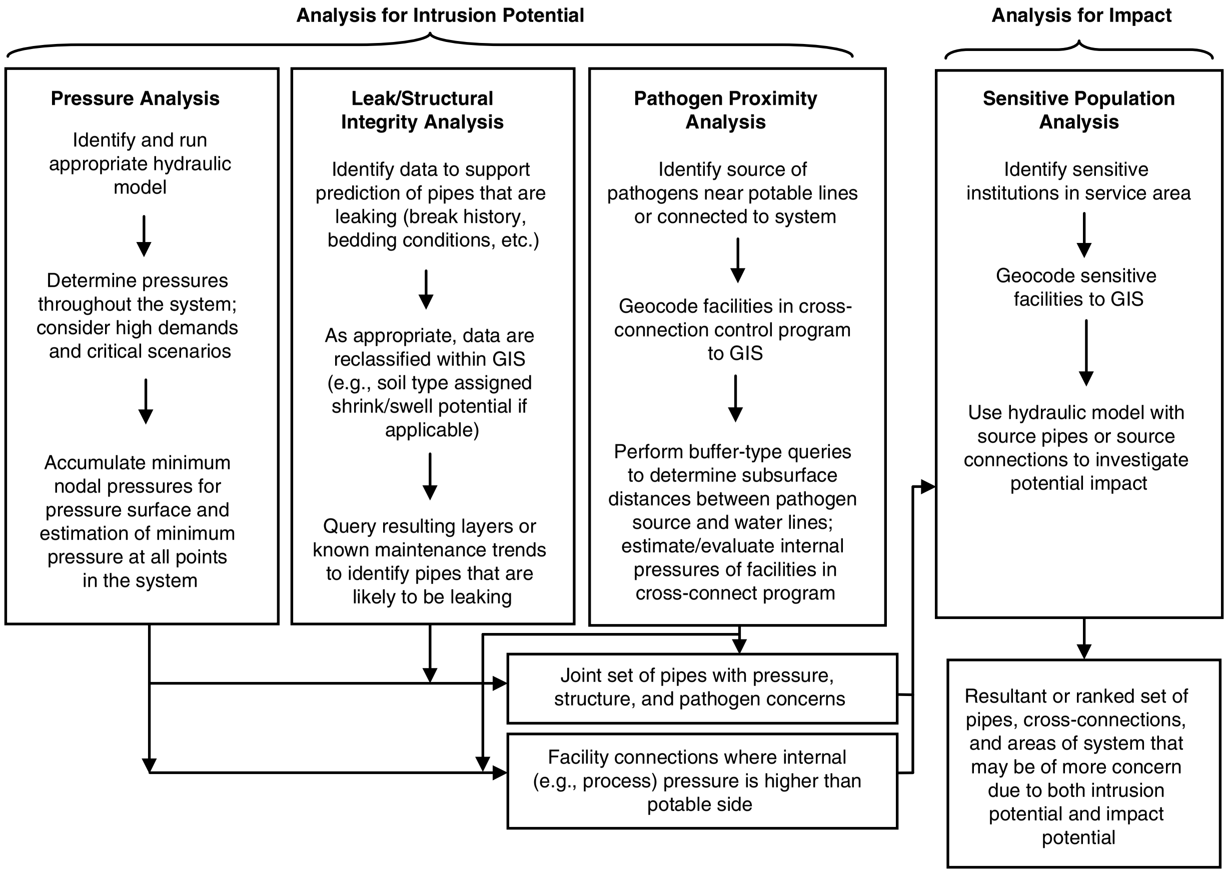

Figure 1 Decision framework

to a utility and from those data assess (estimate) the structural integrity of the buried pipe. Assessing the structural integrity of buried assets is challenging and uncertain. However, simply assuming all pipes are not structurally sound does not give a utility credit for past and ongoing repair or rehabilitation activities. Considering this and the increasing availability of data that could support assessments of buried distribution lines, a more rigorous structural assessment is conducted here as part of the framework.

The structural assessment is based on extensive work already done by utilities and research organizations (e.g., AwwaRF) to assess future needs based on past experience with leaks and breaks. The work here does not necessarily add to the knowledge base relative to assessing infrastructure condition. It is intended to synthesize existing knowledge in support of the objectives, namely identifying segments of the distribution system susceptible to intrusion events.

The structural integrity assessment assumes that the mechanisms leading to pipe leakage are similar to mechanisms that lead to pipe breakage. This assumption is stated explicitly because many references that discuss the condition of drinking water infrastructure are in the context of water main breaks or water main repair prior to a break. Variables typically identified that influence a buried pipe's structural integrity are age/vintage, pipe material, pipe diameter, operating pressure, surface loadings, surrounding soil (expansive or corrosive), proximity of other breaks on the same line, and slope stability. Additional information available to a utility that is a manifestation of these variables includes break and repair histories and leak detection results.

Use of all structural variables is not required to implement the framework. Utilities may opt to use only one or two of the structural variables. For example, a utility may have strong historical evidence that a certain pipe vintage is very susceptible to breakage and leakage while others are not. In this case, vintage alone may be enough to estimate whether the buried assets are leaking or not leaking. Other utilities may observe strong correlations between pipe breaks and bedding conditions; in this case, surrounding soil information may again be sufficient to assess the structural integrity of the system. The basis for which variables should be considered is utility-specific.

Pathogen proximity analysis

Any type of intrusion to the system is a concern; however, pathogen intrusions are of much higher concern. The pathogen analysis involves investigating the proximity of drinking water pipe segments (which have both pressure and structure concerns) to a pathogen source. Pathogen sources include sanitary, storm, or combined sewer lines; septic systems; cross-connections; and other locations where the distribution system may experience pathogens (e.g., river crossings). The framework does not address the integrity of the pathogen sources. It only provides decision logic relative to the proximity of drinking water lines to those potential sources. The basic assumption is that if the separation distance criterion is met, then pathogens at the drinking water pipe are possible. The actual distance criteria for determining if a pathogen source may impact a pipe segment are utility-specific but could be based on knowledge about pathogen transport in the subsurface, the reality of utility construction practices, and any knowledge about the types of possible cross-connections in a given area.

Sensitive population analysis

The demographic impact analysis involves locating (by geocoding within the GIS platform) the sensitive populations relative to the distribution system and determining if water passing the pipe segments of concern could flow to the sensitive populations. The demographic analysis will be topological in nature; it will consider flow paths from the possible contamination source (pipe segment) to the population of concern. This analysis is most easily carried out with the hydraulic model. Clark (2000) has used EPANET to investigate a salmonella outbreak; Maslia et al. (2000) detail the use of EPANET in exposure assessments. In this analysis susceptible pipes will be modeled as sources of contamination in the hydraulic model, and this contamination is allowed to propagate throughout the system. The propagation of water from the new sources can then be observed at all sensitive population centers. EPANET's trace function estimates what percentage of water originating from one node is demanded at a distant node. The duration of the percentage is also available from this analysis.

Ranking

Pipes that are potential sources of pathogens and that impact the populations the most will be targeted for attention. The impact is measured in terms of percentage of water from the intrusion event reaching a given population and the duration that that percentage is present at the node. Source pipes yielding a longer and higher percentage exposure to a given sensitive institution will be ranked higher for attention (e.g., replacement or monitoring). This criterion is also utility-specific.

Implementation of the framework does not explicitly implicate pipes that are routinely being contaminated. It merely synthesizes all of the known risk factors into a coherent framework to identify those areas of the system that are at higher risk from an intrusion than other areas.

CASE STUDY

Application of the decision framework has been preliminarily tested on a study area of an actual utility. The area was chosen on recommendation of utility personnel due to its size, distribution of old and new lines, and compactness. Table 3 summarizes some of the information about the study area. Table 4 compares the skeleton hydraulic model and the reality of the pipe length in the GIS (actual length in use in the study area).

An item of note is the pipe length comparison between the skeleton hydraulic model and the actual pipes in the system. The utility GIS contains all buried facilities down to service connections, including many pipe records that are 6 in. or shorter. However, as seen in the table, the hydraulic model does not offer the same coverage, especially in the 6-in. line. No effort at this time has been made on the part of the authors to rigorously verify calibration of the utility-provided hydraulic input files that are run on EPANET. The model has been used recently by the utility for planning purposes. The input files as provided are assumed to be adequate for this work; this research is essentially a proof-of-concept relative to the approach outlined above. More work may be needed to determine whether the decision support framework is more robust or more automated as the hydraulic model approaches the actual representation of the system (all-pipes model).

The GIS software used for the analysis is ArcView version 3.2, from Environmental Systems Research Institute (ESRI). ArcView was selected primarily due to the availability of ArcView shape files already in existence at the utility. The authors also utilized MS Access as a means to facilitate some of the data transfer between EPANET and ArcView.

Results

The following figures and discussions typify the results currently under review by the authors. The figures are a combination of actual results and expected outcomes and focus on intrusions possible at a location along the pipe wall.

Table 3 Study area

Attribute	Value
Total length of all water lines in the GIS	284.2 miles
Area served	412 square miles
Number of service connections	219,059
Percentage of pipe that is cast iron, ductile iron, concrete, or other within the GIS	58, 36, 4, 2

Table 4 Comparison of length in GIS and skeleton model

Pipe Diameter	Miles in GIS*	Miles in Skeleton Model	Percent of Diameter Represented in Model
6 in.	89.1	10.7	12%
8 in.	138.1	88.4	64%
12 in.	27.7	24.1	87%
>12 in.	26.7	26.7	100%

*Does not add to 100% of total due to some 10-in. and <6-in. pipe not shown

Pressure outcomes

Figure 2 shows the distribution of minimum pressures for all nodes in the system at all times during a 24-hr extended period simulation (approximately 412 modeled nodes). The results are based on demands equivalent to a maximum-month, average-day scenario. This type of demand is typical of a high-use summer month. Figure 2 suggests that under these demands for this area, only 3% of the modeled nodes experience pressures lower than 29 psi. Intrusions at some location along the pipe wall are highly unlikely. The value of 29 psi is equal to approximately 67 ft of water head, suggesting that a pipe operating at 28 psi would have to be submerged under 67 ft of water to have an intrusion at some location where the structural integrity is compromised. A value of 5 psi equal to 11.5 ft of water head is a likely cutoff where the pressure in the pipe is becoming so low that intrusions could occur. However, pipes operating at any pressure are susceptible to backflow if cross-connection control

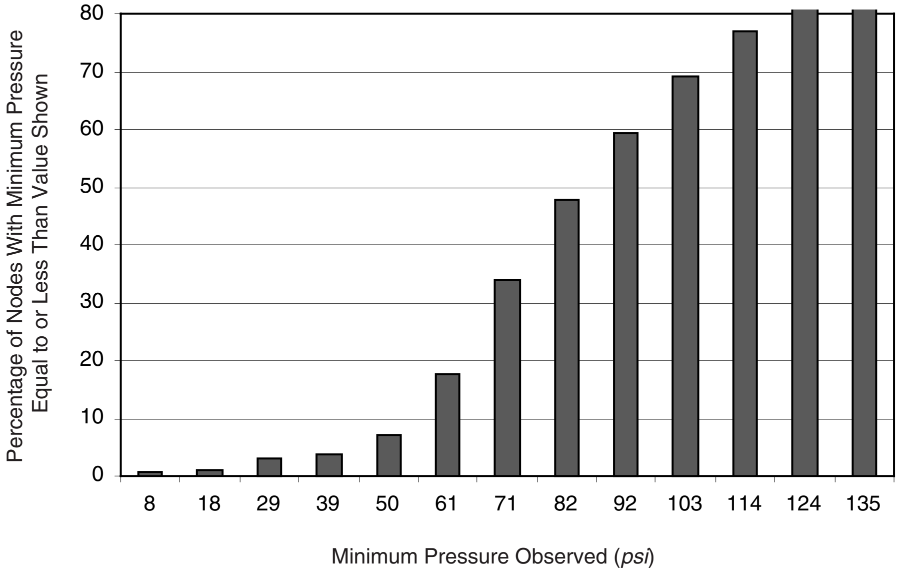

Figure 2 Minimum pressures, all nodes, all times

devices fail and the internal user pressure exceeds the pressure on the potable side. This suggests that areas of the system operating at lower pressures are more at risk to backflow than areas of the system operating at higher pressures. Future analyses will utilize EPANET's Toolbox to implement mid-simulation changes to the network, namely pipe breaks, to understand how this affects pressures throughout the system.

Leak outcomes

The leak analysis seeks to identify factors that contribute to leakage. This investigation was done primarily with GIS data layers and pipe maintenance histories. Figures 3 through 7 show the structural risk factors analyzed. The results of this analysis suggest that for this area, diameter, material, and vintage play important roles in pipe breaks. For example, in Figure 3, approximately 31% of the total pipe length is 6-in. diameter, but over 55% of the breaks occur at this diameter. Figure 6 and 7 are interesting, suggesting for the area under consideration that slope and soil type do not seem to strongly influence the break rate (except for possibly a single soil type). These breakage risk factors are assumed to be leakage risk factors in subsequent analysis.

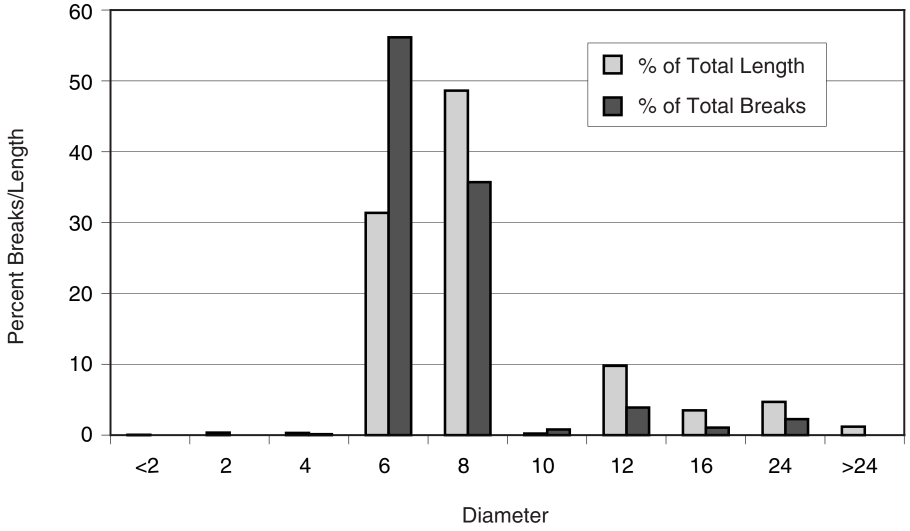

Figure 3 Percentage of breaks by diameter

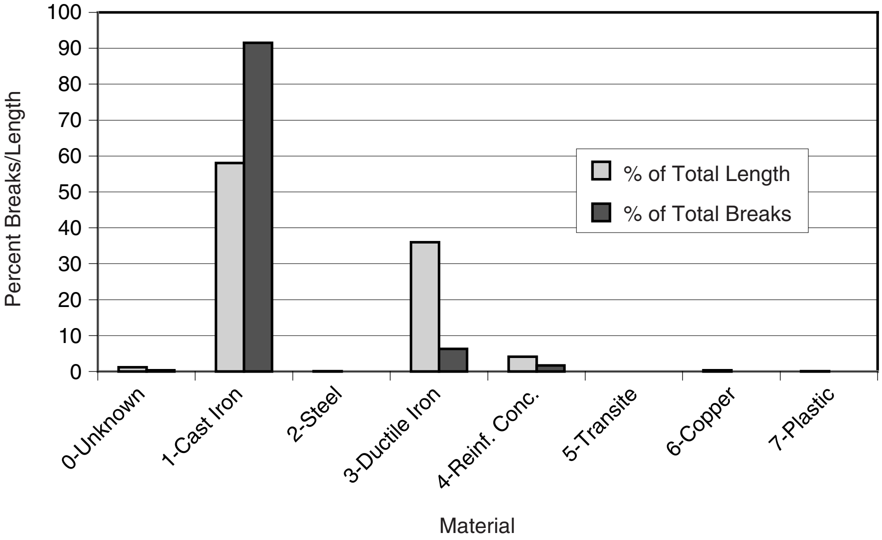

Figure 4 Percentage of breaks by material

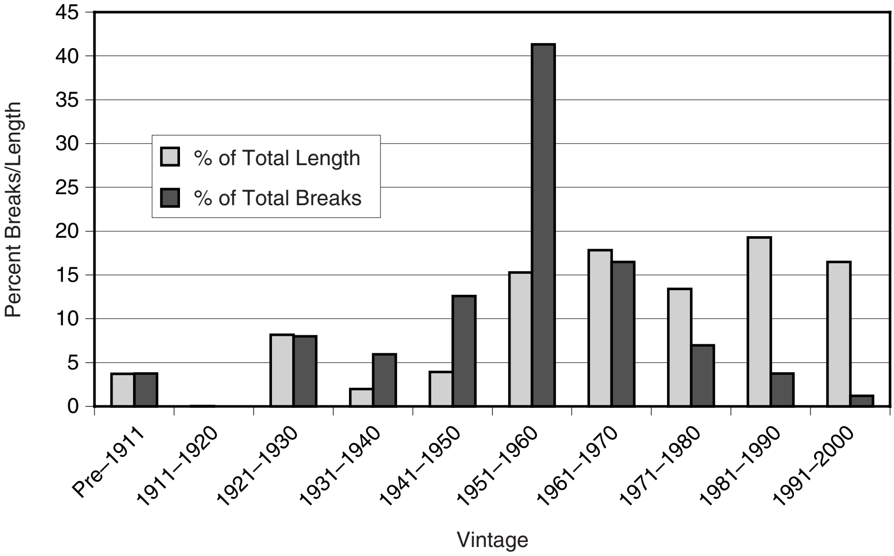

Figure 5 Percentage of breaks by vintage

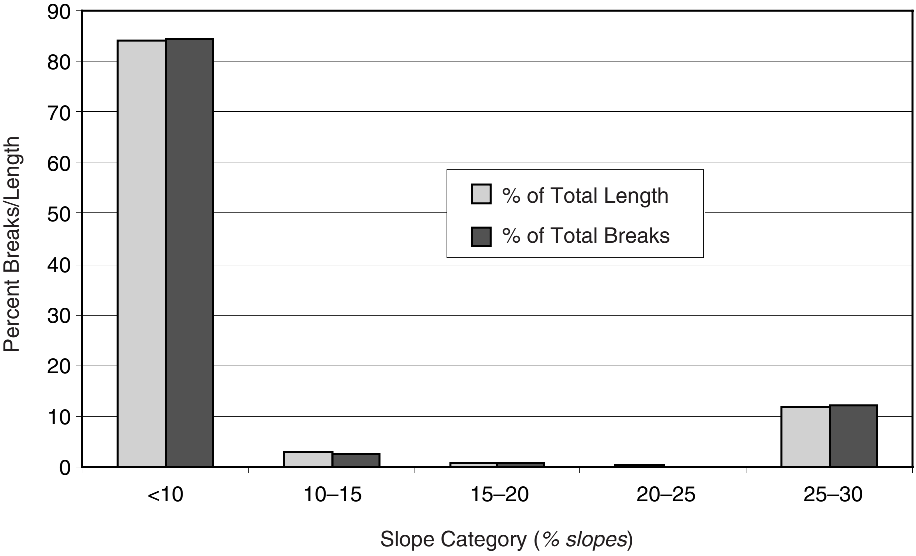

Figure 6 Percentage of breaks by slope

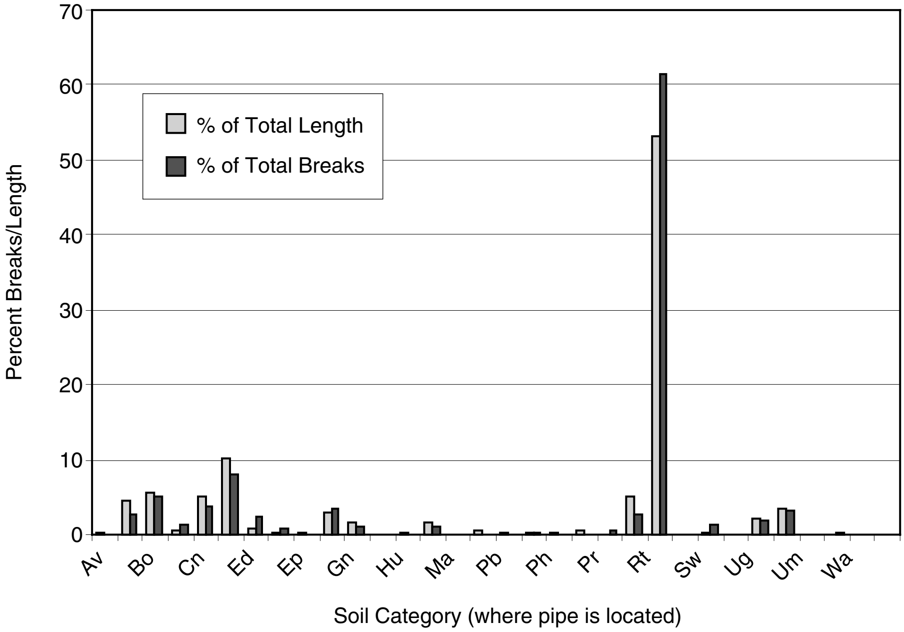

Figure 7 Percentage of breaks by soil type

Pathogen outcomes

The pathogen sources currently reviewed are sanitary sewer lines and septic systems. Distance criteria were analyzed, comparing the lengths of potable water lines within some distance of a pathogen source. Figure 8 shows the results of this analysis for sanitary sewer lines. The distance criterion for how close is "too close" can be utility-specific.

Joint set of at-risk pipes

Table 5 summarizes all analyses in a combined set of segments that have all risk factors in common.

At this point in the analysis a set of susceptible pipes has been found. These pipes are now modeled as source pipes in EPANET to determine the percentage of flow reaching nearby sensitive populations. The authors anticipate analyzing all pipes in the study area and generating a susceptibility-impact plot similar to the one shown in Figure 9. The x-axis in the plot is the susceptibility of pipes to an intrusion event. Those pipes on the far right side of the plot are essentially those summarized in

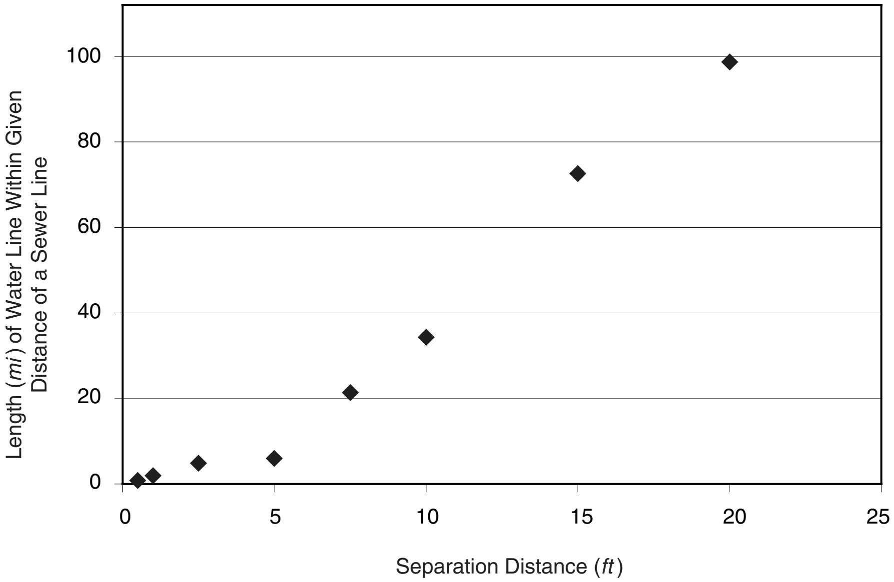

Figure 8 Length of potable line proximal to sanitary sewer

Table 5 Intrusion analysis results

Measure	Pressure Concerns*	Leak Concerns†	Proximity Concerns‡	All Concerns in Common§
Miles of pipe	5	67	6	3.5
Percentage of total	1.9%	24%	2.1%	1.2%

* This value is an estimate based on a preliminary analysis of maximum-month, average-day scenarios. Future work will consider critical events such as pipe breaks, which may increase the footage totals. Pressure cutoff for lengths shown is 20 psi.

† Pipes that are 6-in. and cast iron

‡ Pipes that are within 5 ft of a sanitary sewer line

§ Pipes with all intrusion variables in common

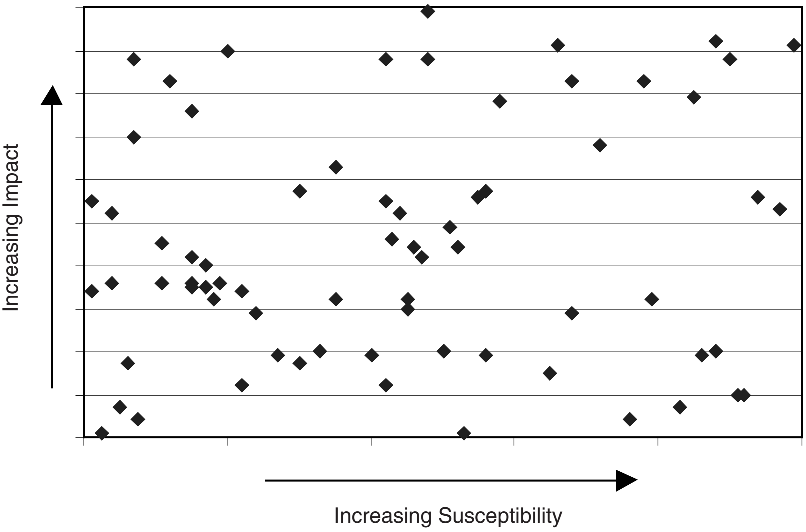

Figure 9 Susceptibility-impact management plot

Table 5. In contrast, a new pipe in a relatively high-pressure zone away from sewer lines would plot at the far left on the x-axis. The y-axis is a result of the EPANET sensitive population (discussed above) analysis where an intruded pipe is a source of contamination. Those pipes plotting highest on the y-axis are relatively close (pipe flow and topology considered) to a source pipe and experience a more significant portion and duration of the intrusion. Those segments in the upper right quadrant are those that have a higher likelihood of both an intrusion and a greater impact given that intrusion does occur. These pipes would receive the first attention in terms of replacement, rehabilitation, or monitoring.

CONCLUSION

Utilities tasked with distribution system management are continually looking for ways to operate their systems effectively given competing constraints. A framework is proposed that provides a total system approach to support both infrastructure decisions and to improve water quality. This approach provides an additional decision variable, namely types of populations served, to help in the difficult decisions of how to repair or replace pipe and where to monitor the system. The framework has been implemented using digital data layers maintained in a GIS and a hydraulic model. This total system approach could be used effectively in some type of accreditation or best management practices protocol. The proposed approach allows a utility to show regulators proactive management of both the system and the sensitive populations, thus minimizing the need for excessive monitoring or regulations.

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CHAPTER · 49

Hydraulic and Water Quality Modeling of Distribution Systems: What Are the Trends in the US and Canada?

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BACKGROUND AND UTILITY CHARACTERISTICS

In October 1999, the AWWA Engineering and Computer Applications Committee (ECAC) commissioned a survey on water network modeling practices. The objectives of the survey were threefold, as listed below:

1. Provide up-to-date information for the revision of AWWA's *Manual of Water Supply Practices, Distribution Network Analysis for Water Utilities* (M32).
2. Gauge the current level of interest and activity in using computers to model water distribution systems, including the behavior of water quality, and to identify potential trends.
3. Identify and share with AWWA members some interesting and possibly innovative applications of water quality modeling in distribution systems.

The survey was distributed to 989 utilities in Canada and the United States. Utilities were selected via a review of AWWA's databases to find utilities serving populations more than 35,000 people. The total number of responses was 174, representing an 18% return rate. The findings of the survey are contained in this chapter. Figure 1 shows the number of utilities in each state and province that responded to the survey. The

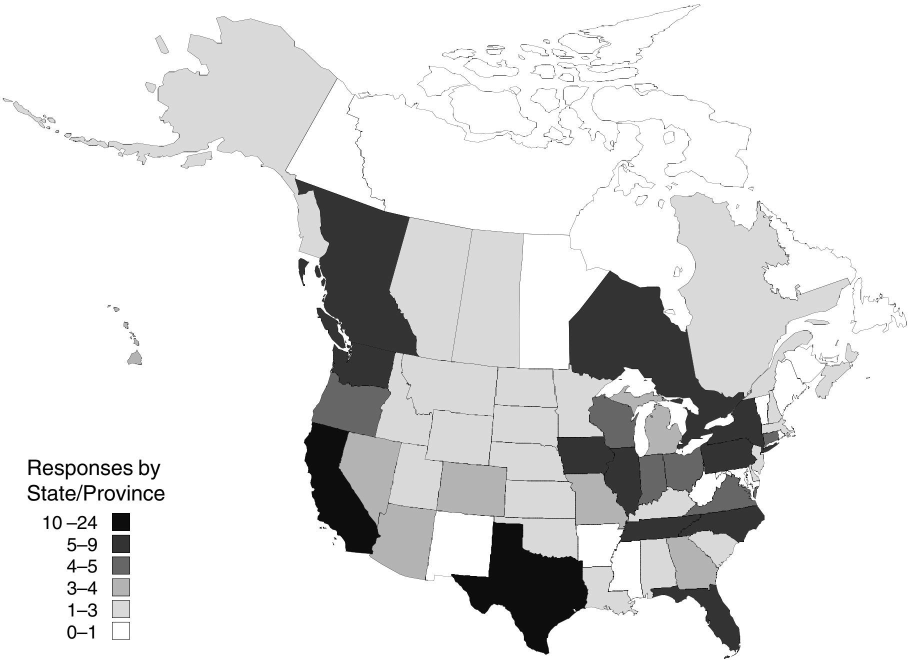

Figure 1 Utility responses

service population of respondents ranged from 21,500 to 8 million people. Figure 2 shows a breakdown of utilities by service population. One-third of the respondents serve between 50,000 and 100,000 people. Almost one-fourth serve between 100,000 and 200,000 people. The remaining size categories had fewer survey respondents. Approximately 68% of the respondents serve a population base of fewer than 200,000. Only 7% of the respondents serve more than 1 million people. Several tables and graphs presented below show the results according to these service population categories to highlight differences, if any.

Service connections, or customers, were also identified. More than 80% of the respondents had 100,000 or fewer service connections. Only eight utilities had more than 250,000 service connections. Figure 3 depicts the range of average service population per service connection, from 0.9 to 2,333 people per connection, ranked in ascending order. All but three respondents had 12 people or fewer per connection, so the top three were excluded from the figure. The very high values are a result of wholesale customers who purchase water through a few service connections but serve a large population. The median value is 3.4 people

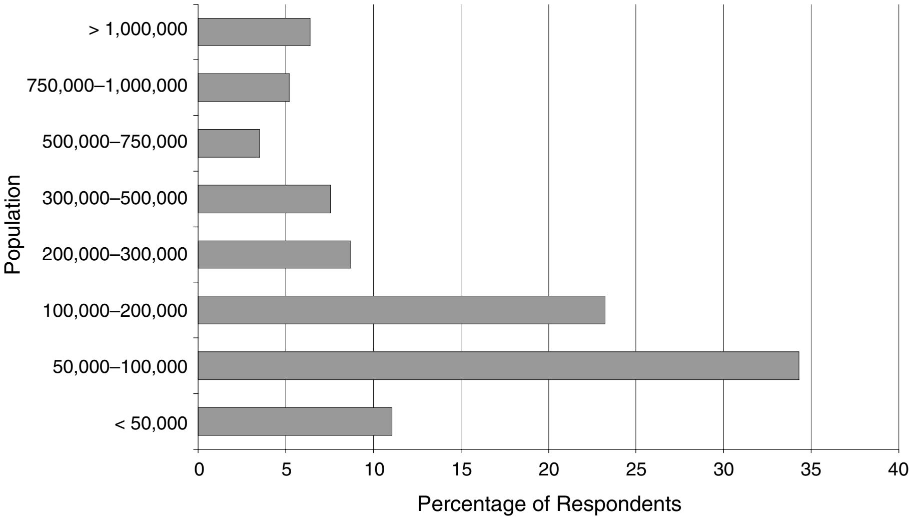

Figure 2 Size of utility

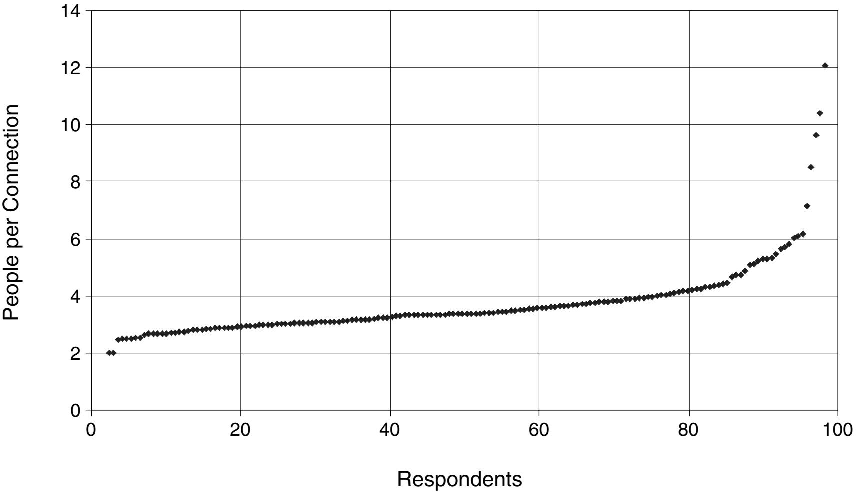

Figure 3 Population per service connection

per connection. Average day demand (ADD) per utility ranged from 2.7 to 1,300 million gallons per day (mgd). The ranges of demands for respondents are shown in Table 1. More than half of the respondents

Table 1 Average day demand

Flow Rate (*mgd*)	<10	10–25	25–50	50–100	100–250	250–500	>500	No Response
Number of respondents	36	61	29	19	14	3	3	9
Percent of respondents	21	36	17	11	8	2	2	NA

(57%) had an ADD less than 25 mgd, and three-fourths of the respondents had an ADD less than 50 mgd.

ADD is plotted in Figure 4, ranked in ascending value of demand per person, and ranging from 75 to 584 gallons per day per capita (gpdpc). The high per capita values are unusual. The 90th-percentile value is 275 gpdpc, which is less than half of the highest value. Climate is certainly one factor causing the high range of per capita demand, because many of the utilities above the 90th percentile are located in the Southwest; however, climate is not the sole factor. Another cause for high demands would be utilities serving commercial and industrial customers that are large water consumers. Modelers who are developing demands based on per capita data must exercise caution and remember to compensate for non-domestic demands that could be a major component of total demands. The median per capital value (50th percentile) was 159 gpdpc.

Source Water Information

Respondents were asked to indicate the source of their drinking water. The responses showed a good cross section covering groundwater, reservoir, and river sources. Figure 5 shows a breakdown of the respondents and types of source water. Forty-four percent indicated more than one type of source water.

The maximum and minimum temperatures for the raw water sources were surveyed because of potential impact on water quality in the distribution system. For example, water temperature significantly affects the decay rate of chlorine residual. More than half of the respondents (57%) observe very high water temperature variations between 30 and more than 60 degrees Fahrenheit in their water supply. Only six of the respondents (4%) have constant water temperatures. Figure 6 shows the

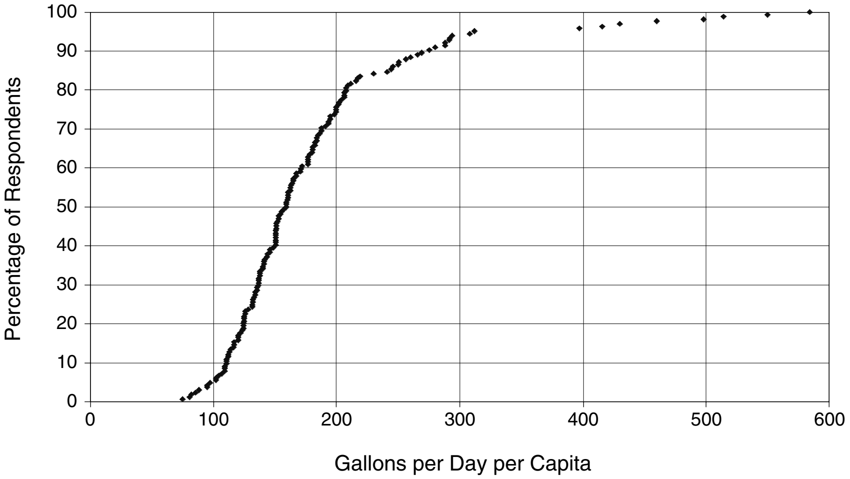

Figure 4 Average per capita demands

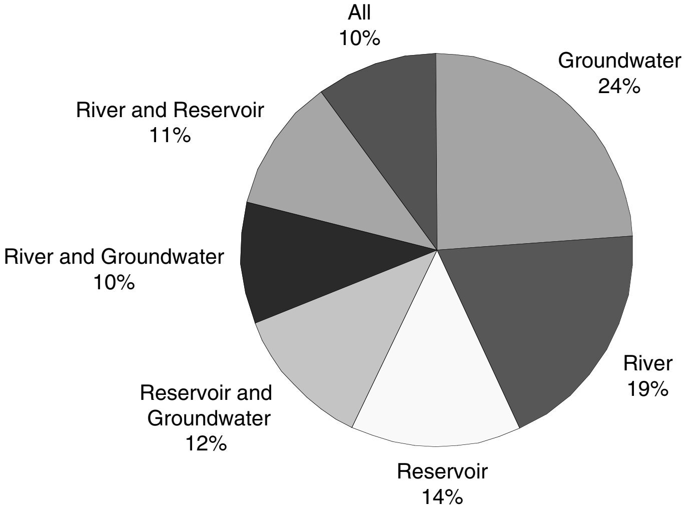

Figure 5 Source water type

minimum water temperature and range of variation for all respondents ranked in order of ascending value of the range. The figure illustrates that water temperature varies for most utilities to the extent that it must be properly addressed when modeling water quality.

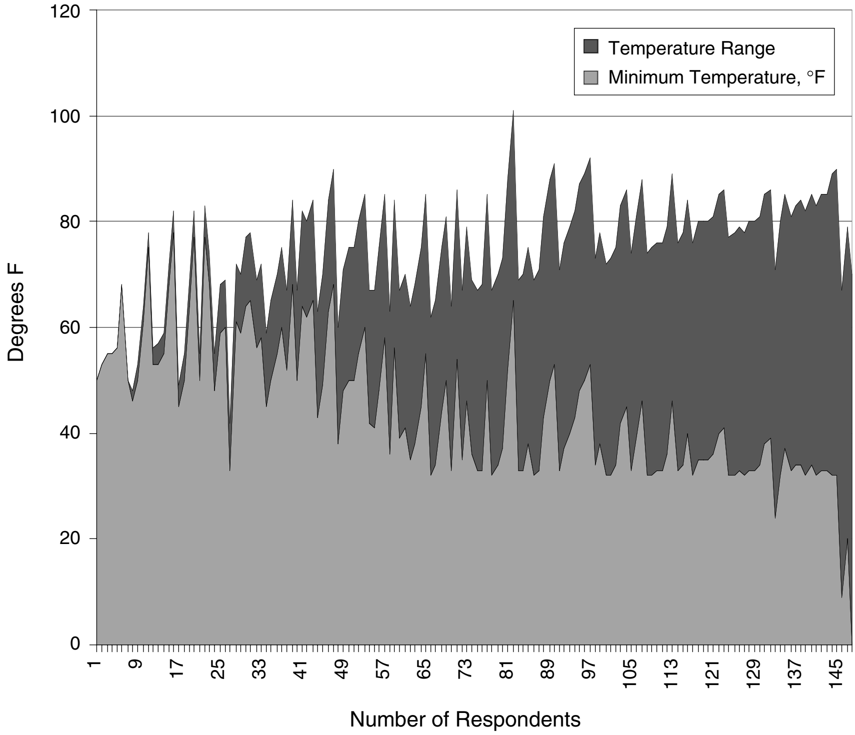

Figure 6 Source water temperatures

DISTRIBUTION SYSTEM INFORMATION

This section contains a series of questions directed toward the characteristics of the respondents' water distribution systems, describing piping, service pressures, storage tanks, and water quality issues—questions often ask about both current and planned conditions. The results are discussed below.

Pipelines

Respondents' distribution systems contained a range of 9 to 12,757 miles of pipeline. The majority of the respondents (69%) had distribution systems with less than 1,000 miles of pipeline, which is depicted graphically in Figure 7. The median value is 460 miles. Figure 8 shows the relationship between the population served and the number of pipelines in the distribution system. Because of some outliers in the maximum and

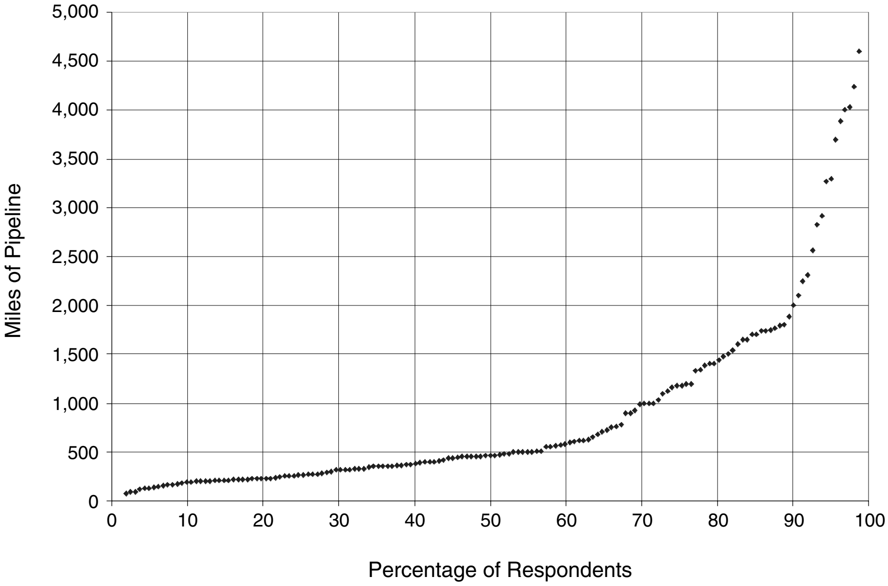

Figure 7 Distribution system pipeline length

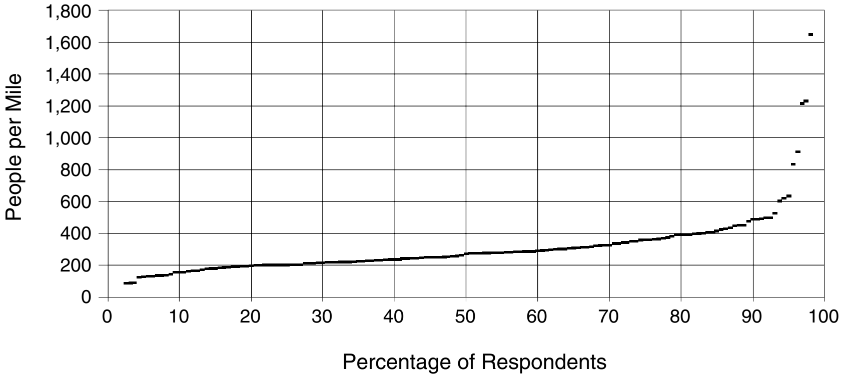

Figure 8 Population per mile of pipeline

minimum values, the top and bottom three points were excluded from the graph. The median value is 270.5 people per mile. Sixty percent of the respondents ranged between 200 and 400 people per mile. These values may be helpful to a utility interested in estimating costs to prepare a geographical information system or distribution system model based on service population, if the length of pipeline is unknown.

The minimum pipe diameter in each system ranged from 1 to 14 in. Three-fourths of the respondents indicated a minimum diameter between 2 in. and 4 in., inclusive. Figure 9 shows the minimum and maximum pipe diameters for each utility arranged in ascending order of minimum diameter.

The maximum pipe diameter ranged from 14 in. to 700 in.; however, the top three maximum diameters were not plotted in the figure because they were much larger values than the maximums given by the other respondents and would inordinately skew the graph. Three-fourths of the respondents indicated a maximum diameter between 20 in. and 54 in.

Pressure Zones

The number of pressure zones varied substantially, from a single zone for 17 respondents to over 100 pressure zones for two respondents. More than half of the respondents had five or fewer pressure zones. Almost half of the respondents (46%) reported between 5 and 25 pressure zones. Table 2 shows a breakdown of pressure zones.

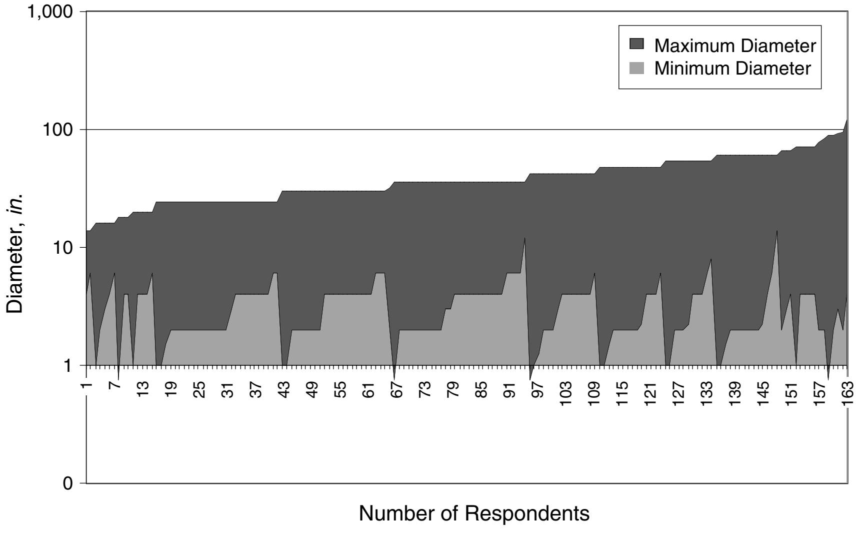

Figure 9 Pipe diameters

Service Pressures

Figure 10 shows both minimum and maximum service pressures for the respondents. The minimum service pressures ranged from less than 20 psi to a high of more than 60 psi. One-third indicated a minimum pressure between 30 and 40 psi. The maximum pressure ranged from a low value of less than 50 psi to a high value of greater than 200 psi. Ten of the respondents had maximum pressures exceeding 200 psi, which is unusually high. Some of the very high service pressures that were

Table 2 Pressure zones

Number of Pressure Zones	<5	5–10	10–25	25–50	50–100	>100	No Response
Respondents	78	44	34	4	5	2	7
Percentage	47	26	20	2	3	1	NA

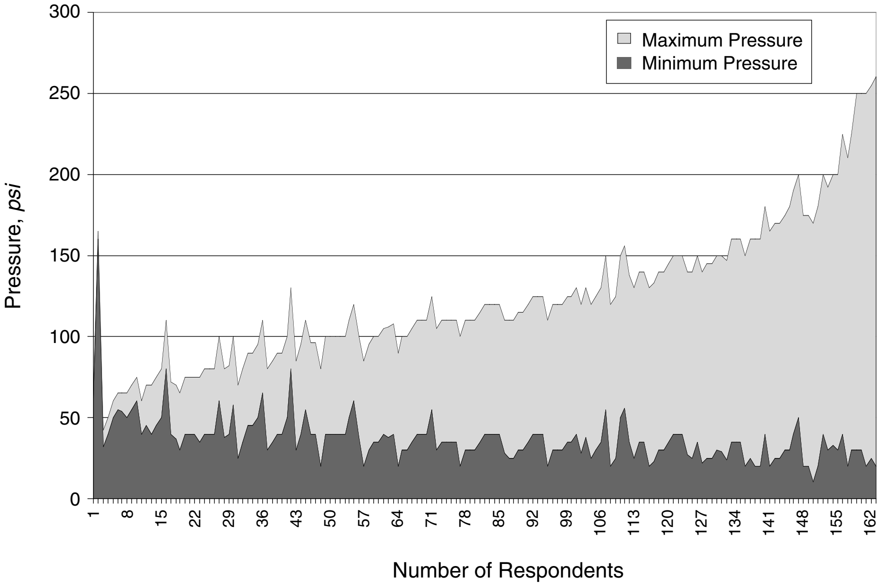

Figure 10 Service pressure

reported may be in transmission mains that do not directly serve customers. About half of the respondents indicated that their maximum pressure was between 100 and 150 psi.

Tank Information

The survey queried the type and number of storage tanks in each system. The types were categorized as fill/draw and inflow/outflow tanks. Fill/draw tanks fill and draw in a distinct sequence, usually because they must be filled or emptied by pumping. On the other hand, inflow/outflow tanks "float" on the system. The two types of tanks are set up differently in distribution system models, and a modeler must be careful to properly simulate their operation. Most of the respondents in Table 3 indicated that they had both types of tanks in their distribution system, but the inflow/outflow tanks were slightly more common.

Secondary Disinfectants

Secondary disinfectants are applied after treatment and remain as a residual in the distribution system. The most common secondary disinfectant used by 89 of the respondents was chlorine. The next most common was chloramine, used by 50 of the respondents, followed by chlorine dioxide used by 6 of the respondents. Figure 11 shows the breakdown of secondary disinfectants used by the respondents.

Table 3 Storage tanks

Number of Storage Tanks	None	< 5	5–10	10–20	20–40	>40	No Response
			Fill/Draw				
Respondents	32	48	46	20	9	1	18
Percentage	21	31	29	13	6	1	NA
			Inflow/Outflow				
Respondents	35	48	27	14	13	5	32

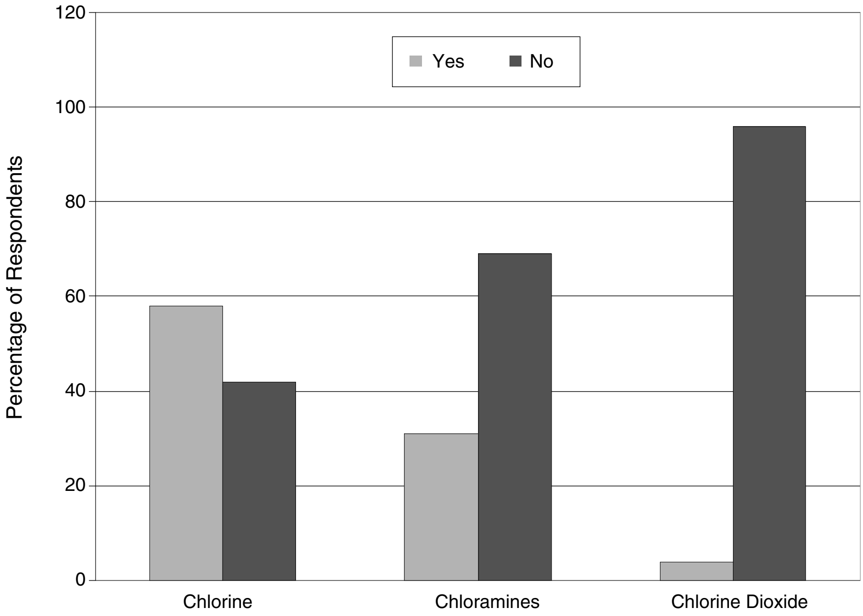

Figure 11 Secondary disinfectant usage

Water Quality Problems in the Distribution System

The purpose of obtaining information about water quality problems was to determine the greatest needs for water quality modeling. Several types of problems were identified:

- Loss of disinfection residual
- Taste and odor complaints
- High disinfection by-product (DBP) formation
- Excessive corrosion
- Coliform occurrences
- High heterotrophic plate counts
- Colored or turbid water

The responses are shown in Figure 12 and categorized by frequency in Table 4. The most common problems cited were taste and odor and turbid water. Only 15 out of 155 respondents indicated that they had no

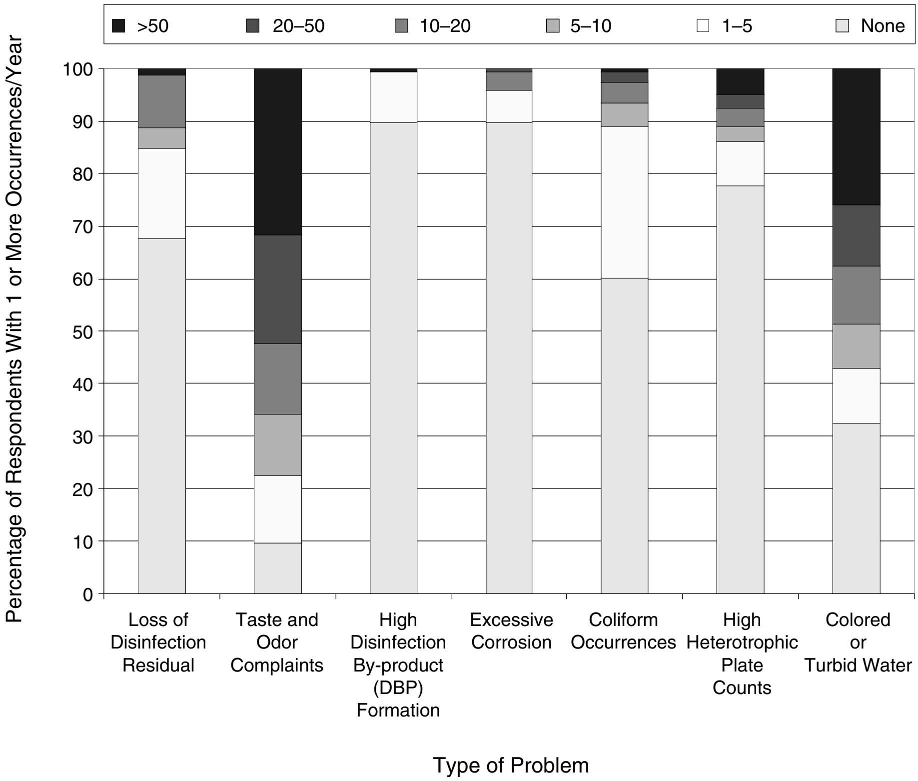

Figure 12 Water quality problems

taste and odor problems, and only 50 out of 154 respondents indicated no problems with turbid water.

The least common problems were high disinfection by-product formation and excessive corrosion. In both cases, 133 of the respondents indicated that these problems did not occur in their systems.

HYDRAULIC MODELING

This section contains a series of questions on the current and planned use of water hydraulic modeling. These questions relate to the following general areas:

- Types and sizes of hydraulic models

Table 4 Water quality problems

	Number of Occurrences						
Type of Problem	None	1–5	5–10	10–20	20–50	>50	Total
Loss of disinfection residual	103	26	6	15	0	2	152
Taste and odor complaints	15	20	18	21	32	49	155
High disinfection by-product (DPB) formation	133	14	0	0	0	1	148
Excessive corrosion	133	9	0	5	1	0	148
Coliform occurrences	92	44	7	6	3	1	153
High heterotrophic plate counts	112	12	4	5	4	7	144
Colored or turbid water	50	16	13	17	18	40	154

- Sources of information for the models
- Calibration information

The majority of respondents (150 out of 174) currently conduct some form of hydraulic modeling at their utility. Ten more utilities, representing an additional 6% of respondents, plan to conduct some form of hydraulic modeling in the future.

Model Information

A query was made to determine the current and planned types of modeling in use or being considered. The responses showed that currently, most utilities have models of their transmission and secondary mains and predominantly do steady-state analyses of their distribution systems. Figure 13 gives some indication of the sizes of utilities that answered the water hydraulic model questions. The size of the utility, as measured by its customer base, should be considered when reviewing the model information presented in this section. It is apparent from Figure 13 that the majority of respondents (more than 50%) provide service to a population of 50,000 to 200,000.

Figure 14 reflects the type of model used in terms of actual percentage of system modeled (skeletonized) and also time step, steady-state versus extended period simulation (EPS), and model that utilities use. Three categories represent the level of detail in the water system

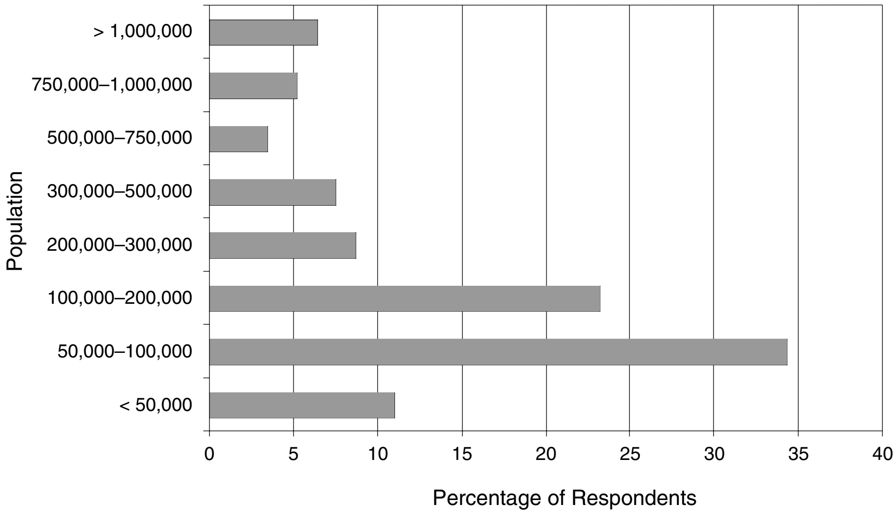

Figure 13 Size of utility

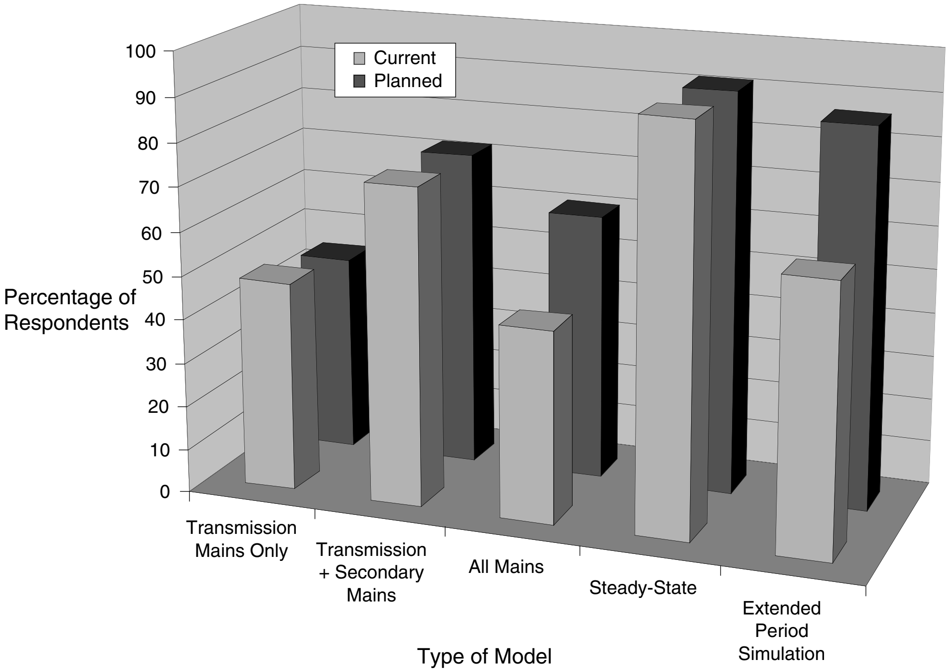

Figure 14 Model types

modeled, beginning with transmission lines (greatest skeletonization) to all mains, which represent all system piping but exclude service level piping. The responses to the question on planned use showed that utilities intend to maintain current uses but indicate an increase in the number of utilities planning to develop all-main models and the development of extended time period models. This indicates that utilities realize the need for more sophisticated models and are creating programs to provide that next level of sophistication.

Figure 15 indicates current and planned uses of the models by respondents. The most common current use of a hydraulic model was in the area of master planning, which is generally a series of evaluations to determine the utility's future system requirements. Fire flow analyses, subdivision planning, and rehabilitation planning were nearly as popular as master planning. Energy management, used to minimize the cost of providing service, was the least likely current use of a model at only 27 of the utilities using this type of analysis. The planned use of the hydraulic models showed continued use of the models for the traditional planning and fire flow analyses and showed a growing desire to use the model for energy management in the future. Clearly, utilities are recognizing the benefit in using the model for operational applications.

Respondents were asked to describe how frequently they use their model based on the various uses shown in Figure 16, which illustrates frequency of use for daily, weekly, monthly, and yearly bases. Most

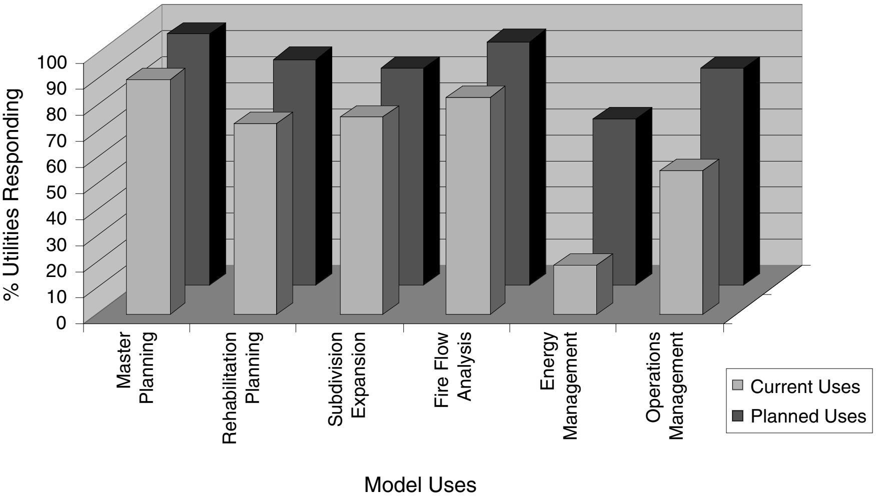

Figure 15 Current and planned model uses

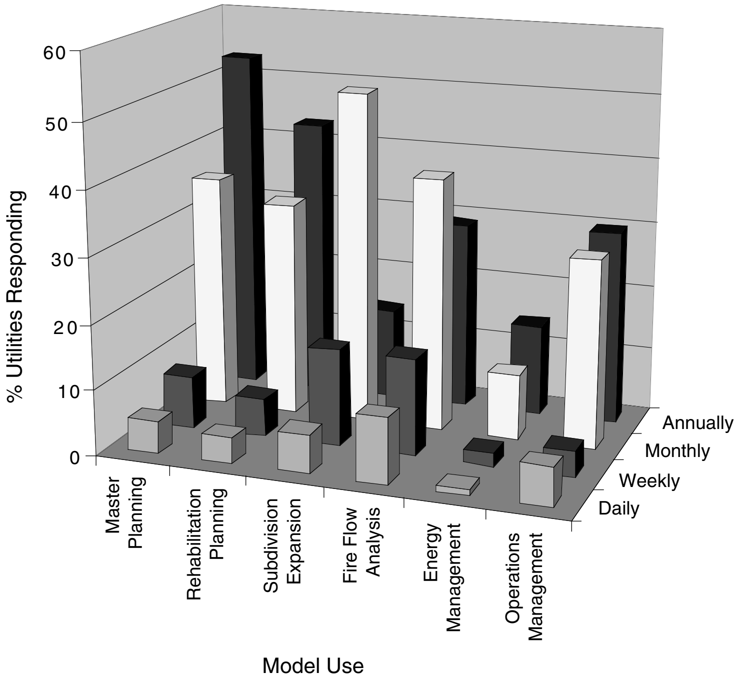

Figure 16 Frequency of use

utilities are using their models on either an annual or a monthly basis. The most common daily use is fire flow evaluations, closely followed by subdivision expansion.

These indicate that models are currently used more by the segment of the utility responsible for assessing water availability for proposed development. These uses follow the more traditional planning activities. As water model use is extended into the operations areas, those areas will likely experience more frequent model use too.

Sources of Model Information

Respondents were asked what source they used for determining consumption and how they allocated that consumption to the model. Figure 17 shows the current and planned sources of data and method of allocation used by percentage of respondents. The most common source of consumption information was the utility billing system (117 of the respondents). Many utilities surveyed did not respond to these questions

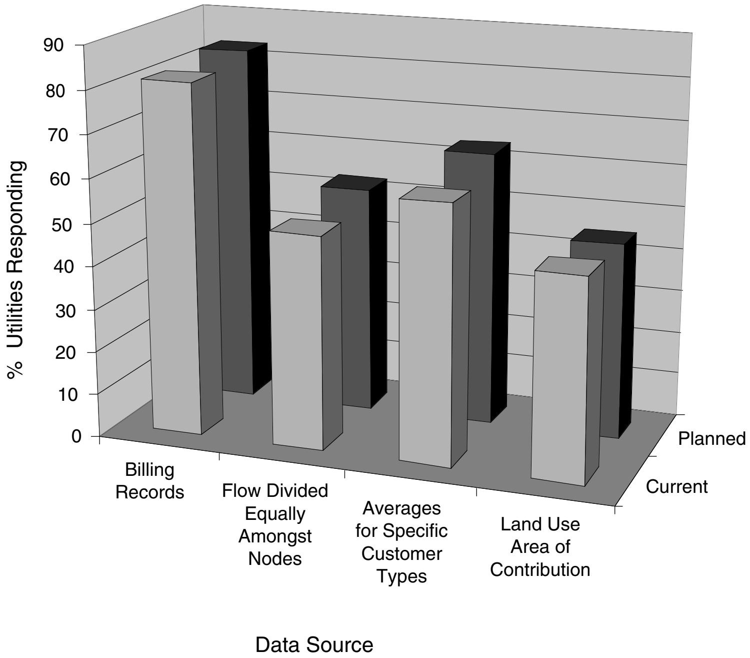

Figure 17 System demand allocation method

on consumption illustrated in Figure 17, especially under the future planned category. Of those that did respond, little difference between current and planned methodologies was noted.

The types of field measurements conducted at each utility were surveyed. Pressure and flow were the most common, followed by consumption and C-factor testing. The least common type of field test was sodium sampling. Figure 18 illustrates types of field measurements currently taken and those planned in the future. It is apparent, based on Figure 18, that utilities are using traditional types of measurements, such as pressure, but are planning to expand their field measurements to include constituents, such as chlorine and sodium, for water quality modeling.

The respondents were asked to identify to which information systems (IS) they are currently linking their models or are planning to link their models in the future. More than any other category in the model portion of the questionnaire, IS applications illustrate dramatic changes planned in the future. Figure 19 shows the findings for IS applications, including AM/FM/GIS and CAD for system configuration

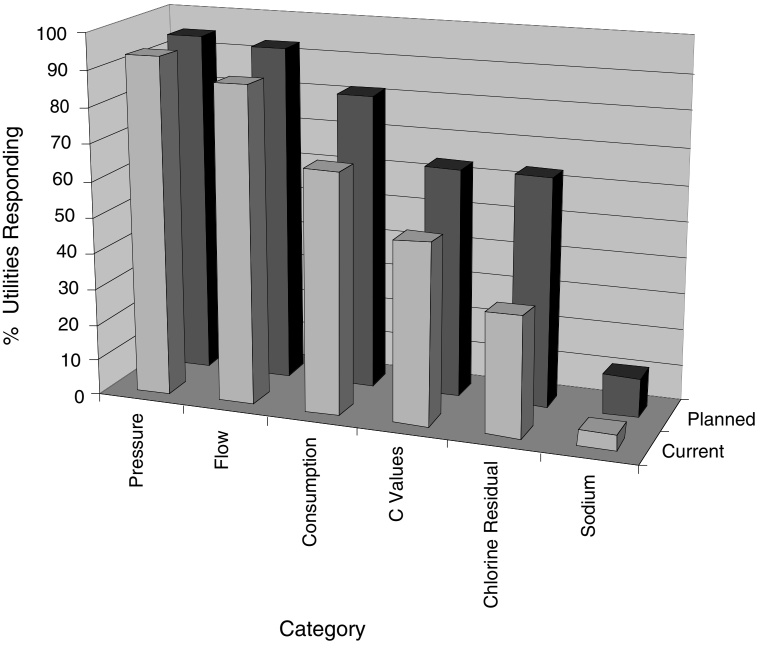

Figure 18 Types of field measurements

and geometry; customer information systems for consumption data; customer complaint data; and SCADA for on-line calibration, operating scenarios, and energy management. Currently, the most common linkage is to a SCADA system (66 of the respondents). It is interesting to note, however, that in all areas considerable growth is expected in the future for development of linkages between information systems and models. The most dramatic anticipated growth is the desire to link to AM/FM/GIS, which increased from 29 to 109 responses. This information confirms the observed desire for an increased level of sophistication in distribution system modeling. As data from other sources are automatically loaded into the water model, the ability to evaluate model performance using real-time data can dramatically improve the model and its accuracy.

Calibration Information

Four questions were asked regarding model calibration:

- Who does/did the calibration of the models?
- When were the models last calibrated?

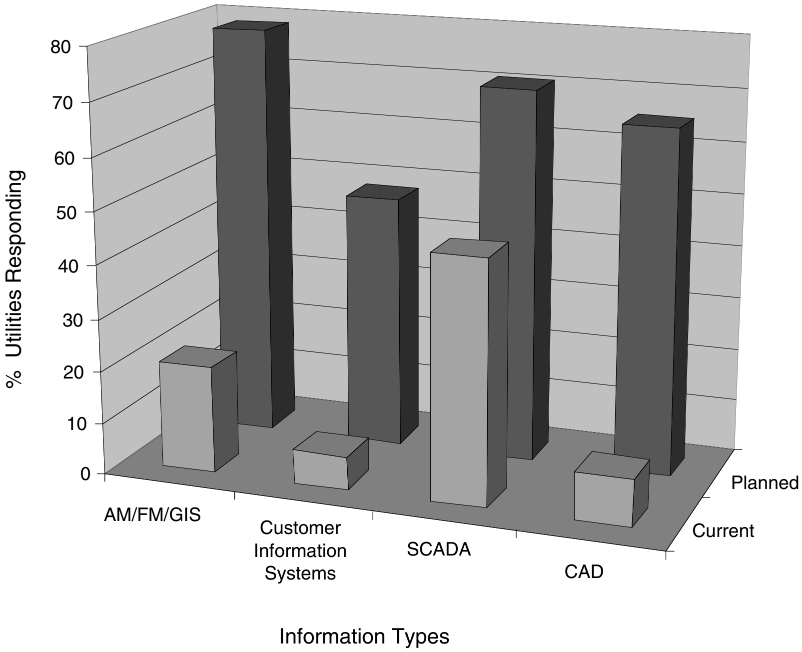

Figure 19 Information systems tied to model

- What is the calibration accuracy?
- What is the calibration frequency?

The majority of the respondents indicated that they used consultants to complete their model calibration efforts and that the most recent calibration was completed within the last four years.

Figure 20 illustrates the amount of respondents that regard their models as calibrated compared with system size. The percentage of respondents that regard their own systems as calibrated ranged from a low of 78% to a high of 100%. However, 55% of the utilities did not respond to this question, possibly indicating that many of the respondents are unsure of the accuracy of their models. Of those responding, the actual definition of calibration accuracy varied. Some established an acceptable deviation from measured pressure (within 5 psi of measured, for example), while others based accuracy on a percentage of the measured pressure. Most of those who responded affirmatively in Figure 20 typically used a calibration standard, such as maintaining predicted

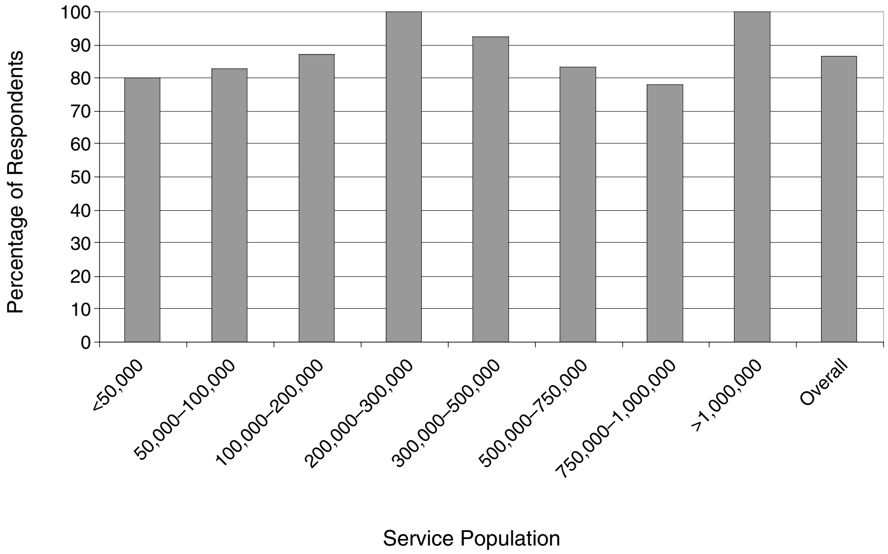

Figure 20 Models that are calibrated

pressure within 5% of actual system pressures. There seems to be no correlation between size of model or system and the degree of calibration.

Similar to the response to accuracy of calibration, the most common response to the question on calibration frequency was again no response (69 of the 174 respondents). Excluding those that did not respond, the majority of utilities indicated a calibration frequency of two to five years, or as needed.

WATER QUALITY MODELING

This section contains a series of questions regarding the current and planned activities in water quality modeling. Based on an analysis of the survey results, 42 of the 173 respondents (24.3%) indicated that they were currently performing some form of water quality modeling. For those planning modeling in the future, the affirmative responses increased to 59%, indicating a substantial growing interest in water quality modeling. Survey respondents were queried for the following current and planned model characteristics.

Constituents Modeled

The survey results show that chlorine and water age were the most commonly modeled parameters for both current and planned efforts. Both of these parameters provide insight into common issues related to water stagnation in the lines, and are related to the presence of tastes and odors in the system and disinfection by-product (DBP) formation. The results also show that there is significant growing interest in future modeling of trihalomethanes, haloacetic acids, and chloramines. The growing interest is likely driven by regulatory compliance issues, and these models will provide a useful tool to evaluate scenarios for converting to new disinfectants or investigating DBP formation potentials. The responses received for this portion of the questionnaire are shown in Table 5.

Degree of Skeletonization

This indicates what fraction of total pipes is included in the model. For this survey an "all-main" model consists of all pipes, or at least all pipe loops within the distribution system. The only pipes that would not be modeled in an all-main model are dead-end mains, which do not affect any circulation pattern and whose demand can be suitably represented at

Table 5 Constituents modeled

	Respondents Replying Affirmatively			
	Current No.	Current %	Planned No.	Planned %
Chlorine	23	13.3	73	42.2
Chloramines	4	2.3	39	22.5
Fluoride	12	6.9	36	20.8
Phosphate	3	1.7	19	11.0
Sodium	4	2.3	14	8.1
Conductivity	5	2.9	23	13.3
Hardness/alkalinity	8	4.6	23	13.3
Temperature	5	2.9	26	15.0
Source blending	19	11.0	43	24.9
Water age	26	15.0	70	40.5
Trihalomethanes	7	4.0	42	24.3
Haloacetic acids	5	2.9	28	16.2
Other	1	0.6	33	19.1

a node on the nearest pipe that is modeled. It is of particular concern in water quality modeling because of the need to ensure that all circulation patterns are included, that the percentage of contribution from each source can be positively identified, and that the smallest pipes with the potentially longest water age are modeled. Figure 14 shows that 44% of those who responded have all-main models. More importantly, the respondents planning to create all-main models in the future increased to 61%, demonstrating that the respondents see the value of all-main models for water quality modeling.

Steady-State Versus EPS

Another issue of interest concerns the use of steady-state versus EPS models. A steady-state model uses demand conditions and operational conditions for a single period or “snapshot” in time. Maximum day, maximum hour, minimum day, and minimum hour are all common steady-state model conditions that are used. An extended period simulation model is a series of snapshot models that have demand and operational conditions that simulate a time period, such as a 24-hour day, by conducting several steady-state analyses at equal time intervals, such as 1 hour. EPS requires operating data to simulate how the demand varies for each hour of the day, and how the utility adjusts operational set points to accommodate greater or lesser demands. The survey indicates that EPS modeling for water quality applications will be increasingly used by respondents in the future.

Purposes or Uses of Water Quality Models

Water quality modeling is currently being used for a wide range of purposes. Seeking operational information is the most common use, but other popular uses are to investigate water age, and to locate and size storage tanks. An example of operational information is to help design flushing programs for improving water quality. Several anticipated future uses were identified and are listed in Table 6. Applications for which significant interest is shown for future use are optimizing disinfection residual, reducing water age, identifying stale water zones, planning flushing programs, and improving operations.

Tank and Reservoir Modeling

Almost all current water quality models simulate storage tanks as completely mixed reactors. This is a simplification for convenience in most commercially available modeling software in order to avoid

Table 6 Water quality modeling applications

	Percentage Responding Affirmatively	
	Current	Planned
Comply with regulations	11.0	34.1
Optimize residual disinfection	10.4	43.4
Optimize corrosion inhibition	3.5	24.3
Assess DBP exposure	4.6	28.3
Reduce water age	13.9	39.9
Identify/mitigate stale water zones	11.6	43.9
Plan/optimize flushing	8.1	45.7
Analyze pipe cleaning and replacement	8.1	34.1
Locate/size storage tanks	13.9	33.5
Improve operations	19.1	47.4
Other	0.6	2.3

computational complexities and unknowns in modeling storage as something besides a completely mixed reactor. However, much research has been performed regarding the water quality in distribution system storage that demonstrates that storage water is far from completely mixed. Uniformity of water quality in storage is highly dependent on a number of factors, including tank shape, inlet and outlet configuration, water temperature, ambient temperature, and average detention time. Therefore, the complete-mix assumption is an oversimplification. As a result, the use of multicompartmented models or other advanced tank representation models will be more prevalent as more is learned about this subject. The questionnaire indicates that only two respondents are currently using a multicompartment reactor model.

Water Quality Calibration

Calibration of any model is a critical issue that determines the validity of the results. The degree of calibration and follow-up verification that is justifiable depends on the intended use. For example, if the purpose of a model is only to provide general guidance on operational issues, the degree of calibration may not be as important. On the other hand, using the model to help predict water quality constituent concentrations or to design new facilities would justify a high degree of calibration.

Table 7 shows an equal number of respondents using historical data and intensive field investigations for water quality calibration. While historical data such as pressures, tank levels, and flow rates can be useful for hydraulic calibration, historical water quality data would be more challenging to use because of the difficulty of simulating historical water quality in an attempt to match the data. For the future many more respondents are planning to perform some sort of water quality calibration. Of the 44 respondents that currently have a water quality model, eight did not calibrate at all, but this is expected to decline in the future, as shown in the table.

The ultimate degree of calibration would be demonstrated when the predictive results of the model could be reliably compared with the real-time data from a supervisory control and data acquisition (SCADA) system. Water utilities that strive for this capability will find many useful applications for optimizing operations and managing water quality in the distribution system.

Table 7 Method of water quality calibration

	Percentage Responding Affirmatively	
	Current	Planned
Historical data	18	49
Intensive field survey	18	36
None	8	4
Other	0	3

CHAPTER · 50

Creating a Planning Tool for the City of Phoenix: Using Field Test Results to Calibrate Hydraulic and Water Quality Models

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Malcolm Pirnie, Inc., Phoenix, Arizona

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Carollo Engineers

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Schmueser Gordon Meyer, Inc.

BACKGROUND

The future Stage 2 Disinfectants–Disinfection By-products (D–DBP) Rule will have a significant impact on potable water systems in Arizona, especially surface water systems. This rule, which focuses on controlling the highest DBP levels in a system, will require many Phoenix-area utilities to carefully evaluate their water distribution infrastructure and operational strategies. A likely requirement of this rule is the implementation of an initial distribution system evaluation (IDSE) to identify DBP hot spots. The city of Phoenix (the City) has already started to plan for the future Stage 2 D–DBP Rule and its IDSE requirements. Carollo Engineers, Malcolm Pirnie, Inc., and others are working to update the City's Water Quality Master Plan to help the City comply with this rule and meet other water quality goals. As part of the planning, the project team is taking the City's existing steady-state hydraulic model to the next level: updating infrastructure data, developing extended period simulation (EPS) capabilities, and calibrating predictions of water age and source water contribution. Outputs of water age and source contribution were used as inputs to water quality models to predict DBP formation in the distribution system. The resultant tool will be used for evaluating planning alternatives, identifying DBP hot spots for the IDSE, and improving system operations.

The project team performed a distribution system tracer test during January 2002 to generate data to further calibrate the City's hydraulic model. This chapter summarizes the execution and results of the fluoride tracer testing program and the application of results to model calibration. Targeted sampling areas were identified, taking system flow paths and historical water quality data into consideration. The testing program required significant coordination between different divisions within the City's Water Services Department and with the Phoenix Fire Department to identify secure sampling locations where autosamplers could be installed. City of Phoenix Fire Department facilities were chosen due to their security and locations throughout the city. In addition to samples collected by the autosampling units for fluoride tracer analysis, water quality samples were collected and analyzed during the study for free chlorine residual, pH, temperature, total trihalomethanes (TTHMs), and haloacetic acids (HAA5). These data were used in combination with water age and source contribution data from the fluoride testing to calibrate water quality models for chlorine decay and DBP formation.

INTRODUCTION

As part of the Water Quality Master Plan Update for the city of Phoenix (the City), Carollo Engineers and Malcolm Pirnie, Inc., are developing water quality and hydraulic models for the City's distribution system. In order for these models to accurately predict changes in delivered water quality due to changes in treatment or distribution system infrastructure and/or operation, they must first be calibrated. This chapter documents the results of a tracer test that provided the consultant team with the necessary calibration data.

The main objectives of this testing were to collect two sets of calibration data:

- Tracer data to allow calibration of the water age and source contribution predictions of the City's Water Model in key zones
- Disinfection by-product (DBP) and chlorine residual data at the treatment plants and in the distribution system during tracer testing to calibrate predictive water quality models

METHODS

General Approach

This tracer study was designed to provide insight into the movement of water through the distribution system. The tracer test used a step-decrease tracer feed at the water sources and then used autosamplers to collect samples at several distribution system locations to track the fluoride decrease through the system. An ideal step-decrease tracer feed and response is shown in Figure 1 with travel time calculated as shown. Breakthrough curves can also be used to calculate source contribution by comparing the observed maximum steady-state concentration with the input concentration.

Tracer Feed and System Operations

The tracer test was performed in January 2002 while the Arizona Canal, which serves two of the City's main WTPs, was off-line for maintenance. During this period, only two of the City's water treatment plants (WTPs), the Union Hills WTP (UHWTP) and the Val Vista WTP (VVWTP), were operating; therefore, the number of input sources to the distribution system was minimized. Also during this period, since the on-line plants are located farthest from the main distribution zones, the water age at many distribution system locations is maximized. In addition to the two WTPs on-line during this time, the Verde Wells were running. To generate source contribution information, no fluoride was fed to the well water. A schematic of the City's operational strategy during the tracer test

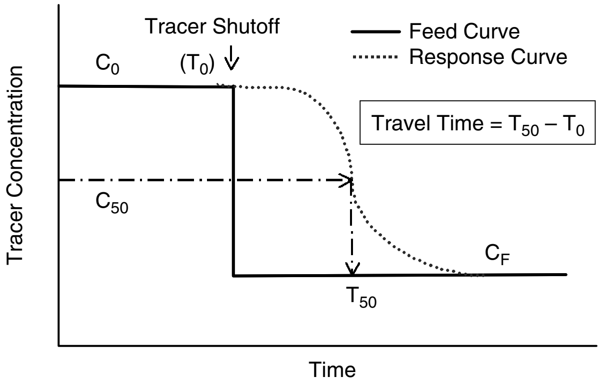

Figure 1 Ideal tracer feed and response curves for a step-decrease test

is shown in Figure 2. Although two areas of the distribution system were tested, the discussions in this chapter are limited to the area affected by the VVWTP.

To establish a near steady-state tracer input, the City's staff increased the fluoride feed at the Rio Salado Feed Station to approximately 1.3 mg/L on January 14, 2002. Several samples were collected from the distribution system sampling points in order for the project team to ensure that a steady-state level of increased fluoride was observed at all locations. On January 24, 2002, the tracer feed was discontinued and intense fluoride sampling began. Tracer sampling occurred at 17 locations throughout the distribution system.

In order to correlate on-line fluoride analyzer readings to laboratory fluoride analysis results, fluoride samples were collected regularly at the Rio Salado Feed Station during the test. Samples were also collected from the Verde Line (at City of Scottsdale Booster Pump Station 74) to facilitate source contribution calculations. Key hydraulic and water quality parameters were also measured and analyzed at the VVWTP during the test.

Distribution System Sampling

Sampling locations

The distribution system sampling locations were selected based on the following factors:

- Located within target pressure zones 1, LW, and 3S
- Historically exhibiting low chlorine residuals
- Historically exhibiting high DBPs
- Located along a flow path needed for model calibration

The targeted pressure zones correspond to areas within the city that are farthest from the WTPs, which are key from a planning perspective due to high water age and DBP formation potential. Ideally, all of the tracer sampling would have been conducted at the City's routine water quality monitoring points to facilitate matching data with existing historical water quality information. However, the intense sampling scheme required the use of autosamplers, and since most of the routine sampling sites are not secure enough to leave sampling equipment unattended, alternate, secure locations were identified. Fire stations were selected as sampling locations because they are distributed throughout the city, are accessible 24 hr/day, and are also secure. In addition to fire

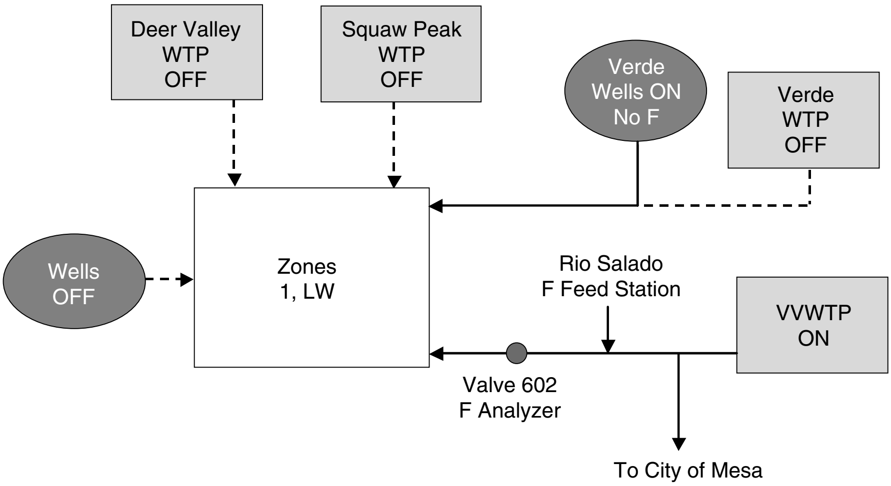

Figure 2 System schematic of operations for the city during the tracer test

stations, some Water Services Department (WSD) sites, including booster stations, wells, and a pressure reducing valve (PRV) station, were selected. The fire stations and WSD sites accounted for 13 of the sampling sites. For the remaining sites not appropriate for autosamplers, the consultant team collected samples manually twice per day. Figure 3 shows the pressure zones, sampling locations, and sampling method (automatic or manual).

Fluoride sampling methods

Autosamplers (ISCO model 3700) were used at 13 of the distribution system sampling locations. Since the autosamplers could not be connected directly to pressurized water lines, a sampling-bucket system was used at each site. Figure 4A depicts a typical sampler setup. In this installation, so that the sink would still be usable, a Y-connection was installed at the tap. Water was run continuously through a length of hose to the sampling bucket from which the autosampler's suction line periodically drew samples. The flow rate was set to achieve a detention time in the bucket of approximately 15 minutes. Water exited the sampling bucket to a drain. The autosamplers were programmed to take discrete samples at 4-hour intervals. Once per day, a member of the consultant team visited each sampling site, collected the samples from the autosampler (Figure 4B), made any flow rate adjustments, and reset the autosampler program (if

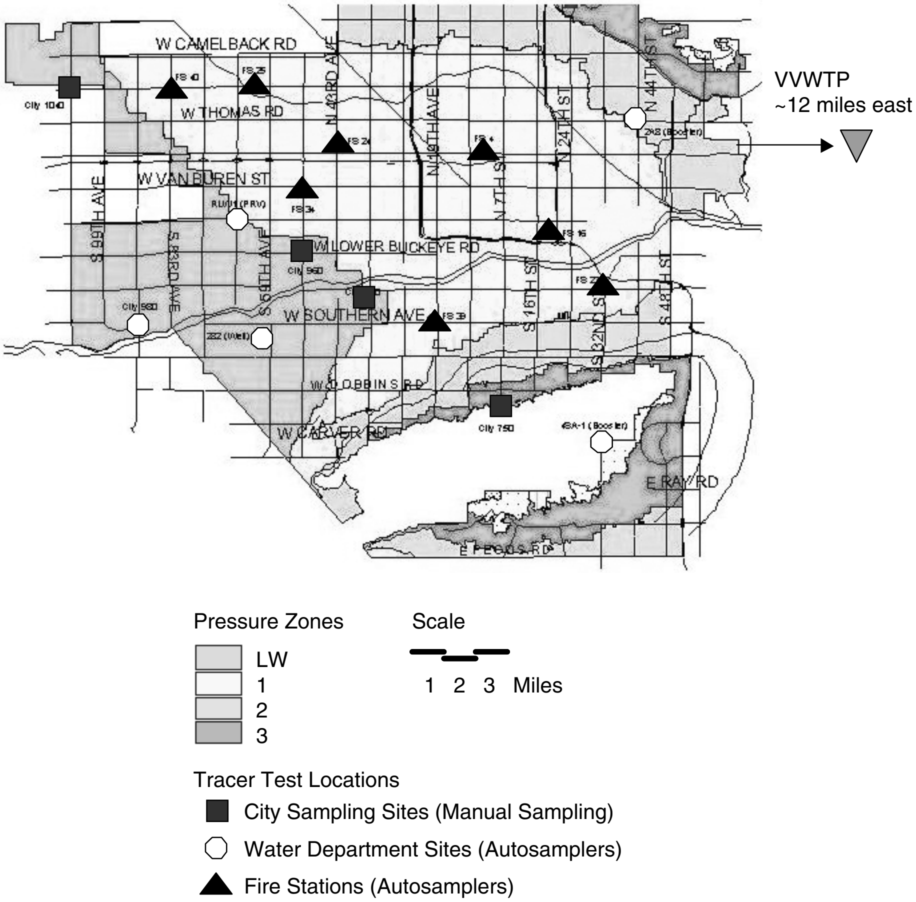

Figure 3 Sampling locations

necessary). Sites where it was not feasible to install autosamplers were visited twice per day as close to 12 hours apart as practical. Fluoride samples were collected manually after purging the sample tap for at least two minutes. Fluoride samples were delivered to the Squaw Peak WTP (SPWTP) and analyzed by the plant chemists according to Standard Method 4500—Fluoride C.

Water quality sampling

In addition to the fluoride sampling, more water quality data were collected on three days during the test. Temperature, pH, and free chlorine

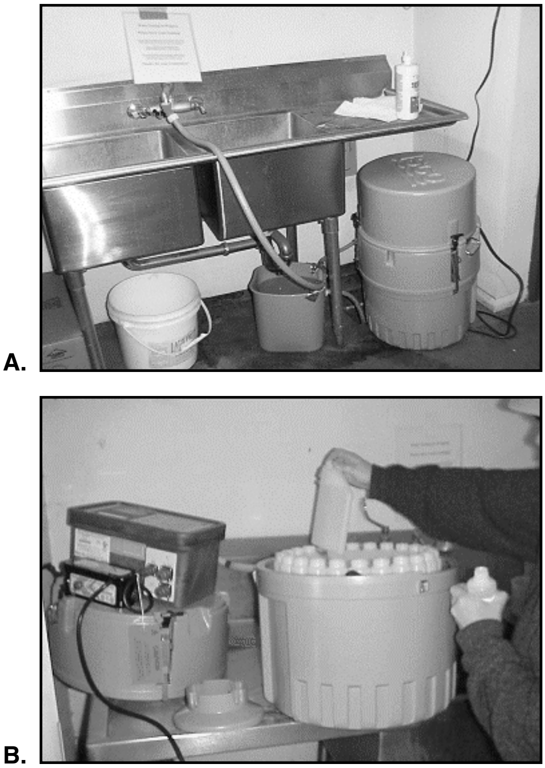

Figure 4 (A) Typical autosampler setup (B) collecting samples

were measured in the field by the consultant team. Free chlorine was measured with an HACH color wheel test kit and the N,N-Diethyl-p-Phenylenediamine (DPD) free chlorine reagent. On these days, the consultant team also collected DBP samples. Total trihalomethane (TTHM) samples were collected in 40-mL amber vials provided by the city of Phoenix Water Quality Monitoring and Compliance Laboratory (COP Lab), quenched with ascorbic acid ($C_6H_8O_6$), and preserved to pH < 2 with hydrochloric acid (HCl). Two TTHM vials, headspace-free, were filled at each location. Water samples for HAA5 analysis were collected in 250-mL amber bottles and preserved with ammonium chloride (NH_4Cl). One headspace-free bottle was filled at each site. DBP samples were stored on ice and delivered to the COP Lab as soon as practical after collection. TTHMs were analyzed according to USEPA Method 502.2 and HAA5 according to USEPA Method 552.2.

Treatment Plant Sampling

On three days during the tracer study, the following water quality parameters were measured at the VVWTP:

Raw Water	Finished Water
• pH	• pH
• temperature	• alkalinity
• turbidity	• TOC
• alkalinity	• free chlorine
• hardness	
• TOC	
• DOC	
• calcium	
• bromide	

Finished water was also collected and stored for simulated distribution system (SDS) DBP tests. The sample vials for the SDS-DBP testing were filled after 0, 6, 24, 48, 72, and 96 hours. TTHM samples from the SDS bottle tests at the VVWTP were collected by plant staff in 40-mL amber vials provided by Montgomery Watson Harza (MWH) Laboratories. Three TTHM vials prepreserved with sodium thiosulfate ($Na_2S_2O_3$) were filled, headspace-free, at each location. HAA5 samples were collected in three headspace-free, 40-mL amber vials, prepreserved with ammonium chloride (NH_4Cl). DBP samples were stored at 4 °C for up to 5 days and then shipped overnight to MWH Laboratories. TTHMs were analyzed according to USEPA Method 524.2 and HAA5 according to Standard Method 6251 B.

In addition, several operational parameters, including the plant flow rates and chemical dosages, were recorded. The water quality and operational parameters were used to facilitate the WTP modeling.

TESTING RESULTS

Fluoride Tracer Results

To compare results from the on-line fluoride analyzers at valve 602 with the fluoride results analyzed by the chemists at the SPWTP, several grab samples were taken in the field at the on-line analyzer locations and analyzed at the lab. The laboratory results differed from the on-line analyzer readings by no more than ±10% with no consistent bias. Therefore, no adjustments needed to be made to correlate the input curves to the breakthrough curves.

A typical tracer response curve is shown in Figure 5. The initial concentration (C_0) at each sampling point was calculated by taking a 48-hr average of the data once an approximate steady-state condition was reached. The final background concentration (C_F) was calculated as the 48-hr average of the fluoride concentration observed by the Rio Salado on-line analyzer after the feed was turned off on January 24, 2002. The calculated travel time shown in Figure 5 is the difference between the time the fluoride concentration reached its midpoint (T_{50}) and the time the tracer was shut off at the Rio Salado feed station (T_0). Figure 6 shows another example of a breakthrough curve and a site with a higher travel time.

In addition to calculating travel time, the distribution system fluoride breakthrough curves were used to determine how much water was coming from a given source. Assuming the fluoride concentration in any

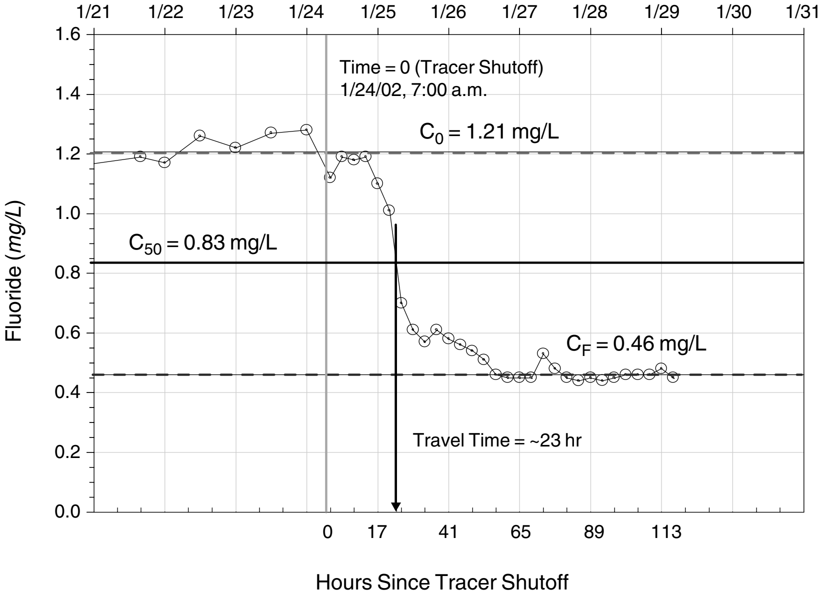

Figure 5 Typical tracer response curve

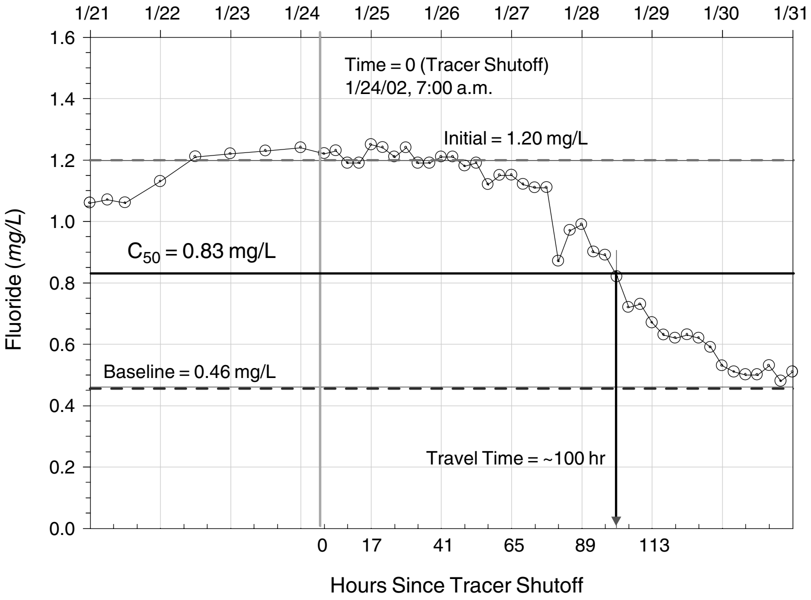

Figure 6 Tracer response curve for site with higher travel time

second sources is known, the source contribution percentages are calculated by solving for *X* in the following equation:

$$C_1 \times X + C_2 \times (100 - X) = C_F$$

Where:

C_1 = Concentration of tracer in primary source
X = Percentage of water from primary source
C_2 = Concentration of tracer in secondary sources
C_0 = Maximum concentration of tracer observed at the sampling point

For the conditions of this study, the primary source was water from the VVWTP and the second source was the Verde Wells. The average fluoride concentration in the well water was 0.41 mg/L. There were two sites that did not reach a steady-state concentration prior to the fluoride decrease, so source contribution could not be calculated. Figure 7 shows the tracer response curve for the site with the most dramatic secondary

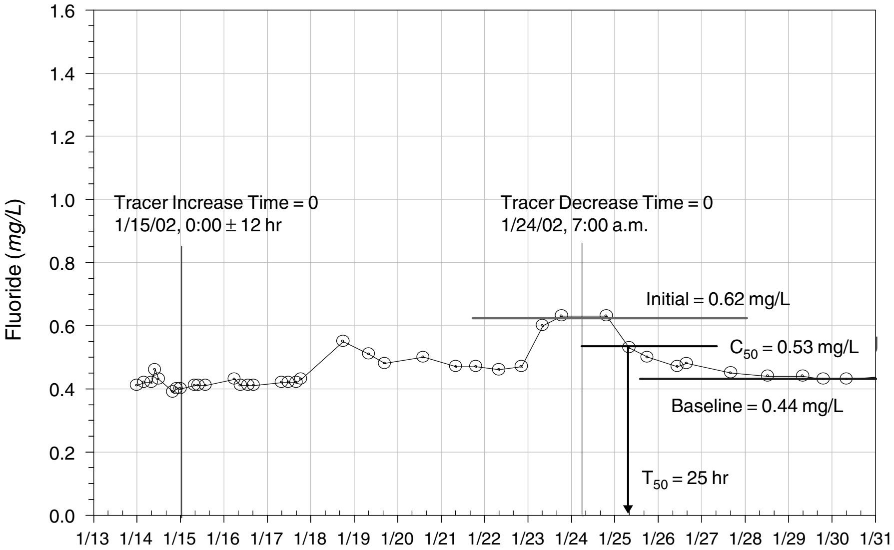

Figure 7 Tracer response curve showing the most dramatic source contribution

source contribution, as the maximum observed fluoride concentration was only 0.62 mg/L.

The travel time shown in Figures 5 through 7 is from the location of an on-line fluoride analyzer at Valve 602, approximately 15 miles from the VVWTP effluent. However, since the travel time of interest is from the plant effluent, 6 hr were added to the calculated time to account for the travel time from the VVWTP to the Valve 602 Analyzer during the test. Figure 8 geographically shows travel time from the VVWTP to each sampling site, and Table 1 summarizes the travel time at each site as well as the source contribution calculation results. It is recognized that the water age at a location can vary with time and that the observed travel time represents the water age at only one point in time.

Water Quality Results (WTP and Distribution System)

On three days during the tracer study, water quality data were collected for raw and finished water at the VVWTP. Water quality parameters were also collected in the field at all distribution system locations on three days. The results are summarized in Tables 2 and 3 as an average of the measurements taken.

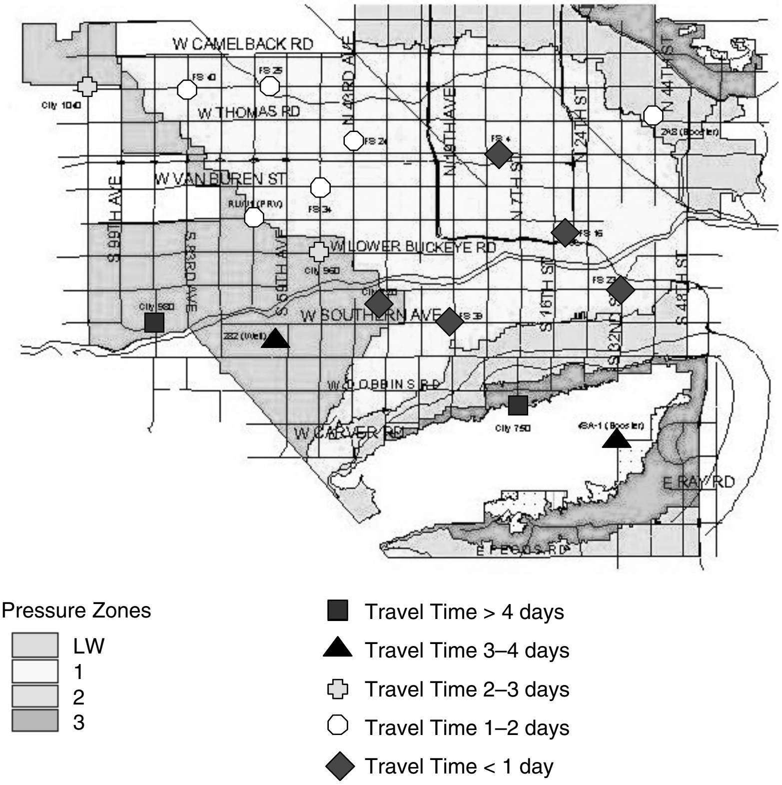

Figure 8 Travel time at each sampling site

In addition to the water quality data, DBP data were generated. At the VVWTP, SDS bottle-point tests were performed. In the field, TTHM and HAA5 samples were collected. By using the calculated travel time from the tracer study, the SDS test results and field measurements can be plotted on the same graph and a comparison can be made. In general, the SDS tests provided a good prediction of DBP formation in the distribution system (Figures 9 and 10). Even though there was an exceptionally long travel time to one of the sampling points, data showed that after 72 hr, the TTHM formation rate was very slow. It is interesting to note that at longer travel times, HAA5 concentration decreased, likely due to biological degradation. Furthermore, it illustrates that distribution system locations with the highest TTHM levels may not necessarily be the same as the locations with the highest HAA5 levels. There is one point showing significantly lower DBP formation. This point corresponds to

Table 1 Travel time and source contribution summary

Site ID	Pressure Zone	Travel Time* (*hours*)	Travel Time* (*days*)	% Treatment Plant Water†
City 750	3S	206	8.6	Inconclusive
91st Ave WWTP	LW	106	4.4	90–98
Well 282	LW	78	3.3	84–92
Booster 4SA1	3S	78	3.3	Inconclusive
City 960	LW	58	2.4	85–100
City 1040	LW	54	2.3	85–95
PRV RLW1	LW	44	1.8	85–98
Station 40	1	38	1.6	83–88
Station 25	1	34	1.4	85–98
Booster 2A8	1	31	1.3	22–26
Station 34	1	29	1.2	87–91
Station 24	1	29	1.2	90–100
City 720	1	17	0.7	85–100
Station 4	1	15	0.6	95–100
Station 23	1	14	0.6	92–100
Station 39	1	13	0.5	91–95
Station 16	1	13	0.5	90–100

*The travel time from the effluent of Reservoir 3 for the VVWTP

† Ranges shown indicate the percentage of water from the treatment plant at all sampling time points for a given location

the distribution system sampling location at Booster 2A8, which received mostly Verde Well water during this study. As expected, DBP concentrations for TTHMs and HAA5 were inversely proportional to the measured free chlorine residual (Figure 11). Although this correlation would not replace DBP compliance monitoring and sampling, it may be an inexpensive method of identifying DBP "hotspots" for the Stage 2 D–DBP Rule Initial Distribution System Evaluation (IDSE). A comparison of Table 2 and Table 3 indicates that pH fell up to 0.4 unit in the distribution system, while temperature rose by up to 5 °C and chlorine residual dropped by 0.5 to 1.8 mg/L. These changes can be expected to be more dramatic in the summer months.

Table 2 Water quality characteristics at the VVWTP during tracer study

Parameter	VVWTP Raw Water	VVWTP Finished Water
pH	8.3	7.7
Temp (°C)	13	
Turbidity (NTU)	4.6	
Alkalinity (mg/L as $CaCO_3$)	152	142
Hardness (mg/L as $CaCO_3$)	221	
Calcium (mg/L)	54	
Bromide (mg/L)	0.10	
TOC (mg/L)	2.8	2.1
DOC (mg/L)	2.6	
UV-254 (1/cm)	0.048	
Free chlorine (mg/L)		1.9
TTHM, SDS 0-hr*		31
TTHM, SDS 6-hr*		43
TTHM, SDS 24-hr*		48
TTHM, SDS 48-hr*		52
TTHM, SDS 72-hr*		61
TTHM, SDS 96-hr*		69
HAA5, SDS 0-hr*		14
HAA5, SDS 6-hr*		16
HAA5, SDS 24-hr*		21
HAA5, SDS 48-hr*		24
HAA5, SDS 72-hr*		26
HAA5, SDS 96-hr*		27

* To allow comparison between WTP and distribution system DBP samples, results from split samples were used to calculate a correlation factor. Data shown incorporate this correlation factor.

Table 3 Distribution system water quality during tracer study

Site	pH	Temperature (°*C*)	Free Chlorine (*mg/L*)	TTHM (*μg/L*)	HAA5 (*μg/L*)
City 750	7.5	12	0.1	71	22
91st Ave WWTP	7.5	18	0.8	55	26
Well 282	7.6	15	0.5	61	24
Booster 4SA1	7.4	16	1.0	60	23
City 960	7.5	15	1.0	51	23
City 1040	7.5	16	0.9	57	24
PRV RLW1	7.4	15	0.9	51	21
Station 40	7.5	18	1.1	53	21
Station 25	7.4	16	0.9	55	21
Booster 2A8	7.6	17	1.2	25	7
Station 34	7.4	15	1.3	48	19
Station 24	7.5	19	1.0	53	23
City 720	7.4	12	1.2	42	18
Station 4	7.6	14	1.4	41	17
Station 23	7.6	15	1.4	42	17
Station 39	7.3	12	1.4	43	17
Station 16	7.6	15	1.1	41	17
Average	**7.5**	**15**	**1.0**	**48**	**21**
Range	**7.3–7.6**	**12–19**	**0.1–1.4**	**25–71**	**7–26**

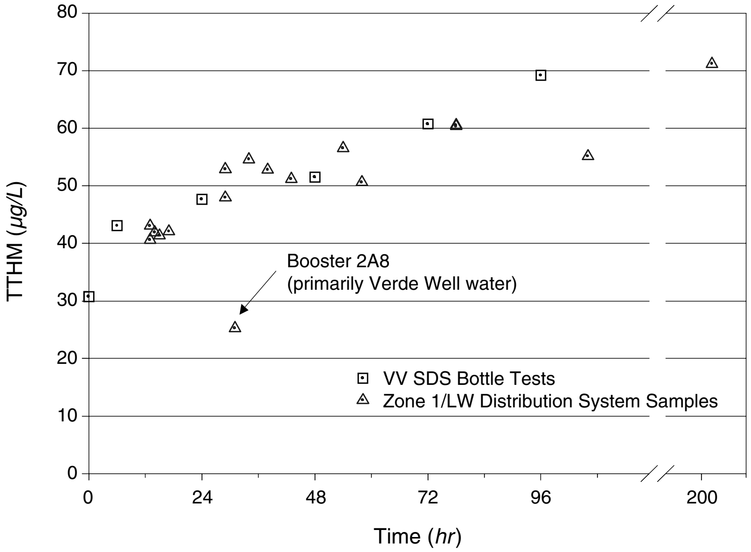

Figure 9 SDS tests are good indicators of TTHM formation in the distribution system

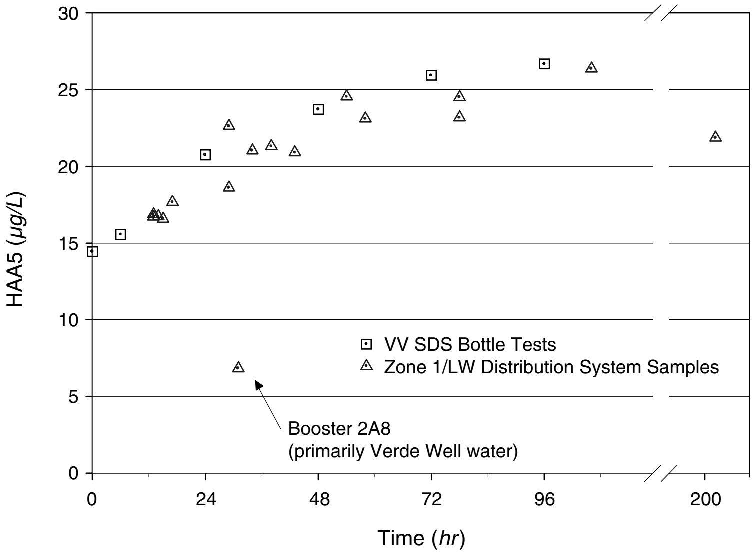

Figure 10 SDS tests are good indicators of HAA5 formation in the distribution system

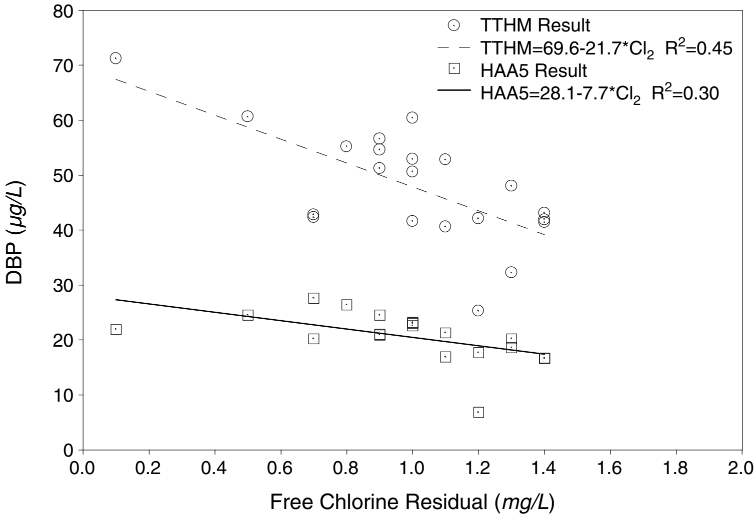

Figure 11 DBP formation as a function of free chlorine residual in the distribution system

Model Calibration Results

The analytical results obtained during the tracer study were used to calibrate two water quality planning tools:

- The extended period simulation (EPS) portion of the City's hydraulic model
- The DBP prediction algorithm in the WTP simulation model

Hydraulic model calibration

Hydraulic modelers at Carollo Engineers used data collected during the tracer study to calibrate the City's hydraulic model. These data included the fluoride tracer response curves, SCADA information (e.g., tank level and flowmeter data), and data from pressure recorders that were installed at several fire hydrants in the distribution system near the tracer sampling points. To determine the demands within the study area during the tracer study, a mass balance analysis was performed on an hourly basis using the City's SCADA information. The mass balance analysis determined that the daily demand in the study area was approximately 59 mgd during the tracer study. The study area included the City's largest pressure zone, Zone 1, and an adjacent pressure zone (Zone LW) that is supplied by pressure reducing valves (PRVs) from Zone 1. Since the PRVs supplying Zone LW were not metered, the precise distribution of demands between Zone 1 and Zone LW was unknown. The demands were allocated using the City's stand-alone "demand generator," which was developed using historic water customer meter data.

The modelers adjusted the following calibration factors: pipe C-values, pipe minor loss coefficients, demands in Zone LW, and loss coefficients associated with the five primary control valves within the study area. The model was run with the actual fluoride feed curve as an input and incorporated the influence of Verde Well water as a secondary source water. The output of the model produced fluoride response curves for each node in the system. The modeled response curves were overlaid on the tracer test response curves for each tracer test sampling point. Two examples of these overlaid curves are shown in Figures 12 and 13, with Figure 12 representing a site where the predicted and observed data match closely and Figure 13 illustrating a "worse-case" site. The model parameters were adjusted within a reasonable range to get the "best fit" to the observed fluoride response curves, system pressures, and reservoir levels at the three floating reservoirs in the study area. The parameter with the largest effect on the modeling was calibrating the losses associated with the primary control valves.

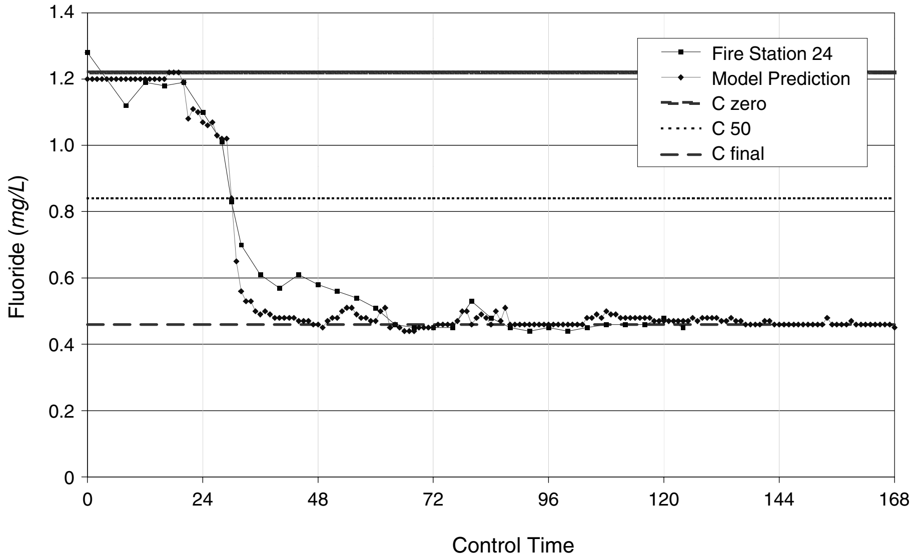

Figure 12 Hydraulic model predictions that closely match the observed tracer data

The travel time predicted by the model ranged from 40 hours shorter to 4 hours longer than the actual observed travel time. The tracer sampling sites that differed by the highest number of hours are also the sites where the tracer test travel time was the longest. Figure 14 shows the difference in hours between the modeled and measured travel times and also indicates the percentage of total travel time accounted for by the difference. At the locations of the largest differences, the differences between the observed and modeled fluoride response curves may be due to difficulties encountered in flushing relatively large-diameter sample collection lines. That is, the distribution line may not have been adequately flushed, resulting in gradual and delayed dropoffs in the observed fluoride concentrations, as possibly shown in Figure 13.

DBP formation algorithm calibration

Engineers at Malcolm Pirnie, Inc., used the water quality and operational data collected at the treatment plant as well as the DBP analytical results collected during the tracer study to calibrate the DBP formation algorithms in the WTP model. The raw water quality parameters measured at the plant for specific days during the study were used as inputs to the WTP simulation model. The plant was simulated using the

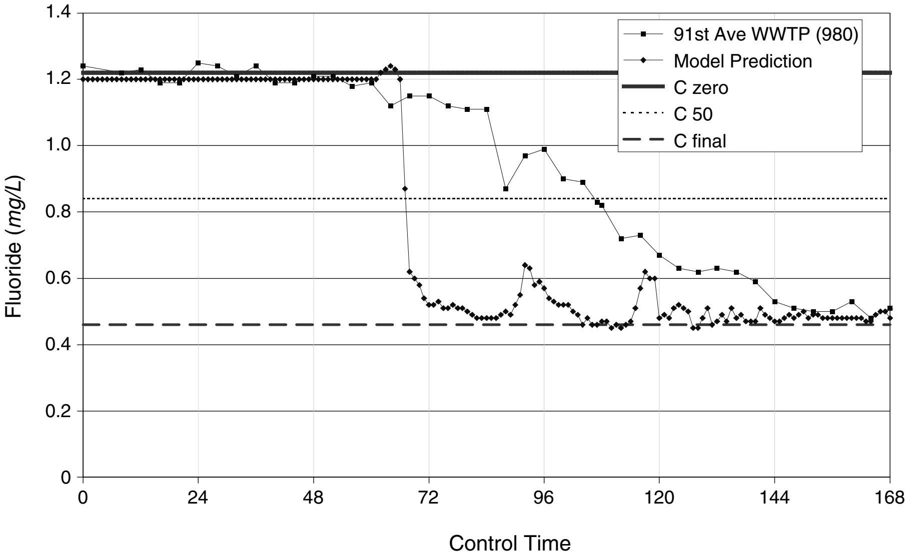

Figure 13 Hydraulic model data that do not match the observed data as closely

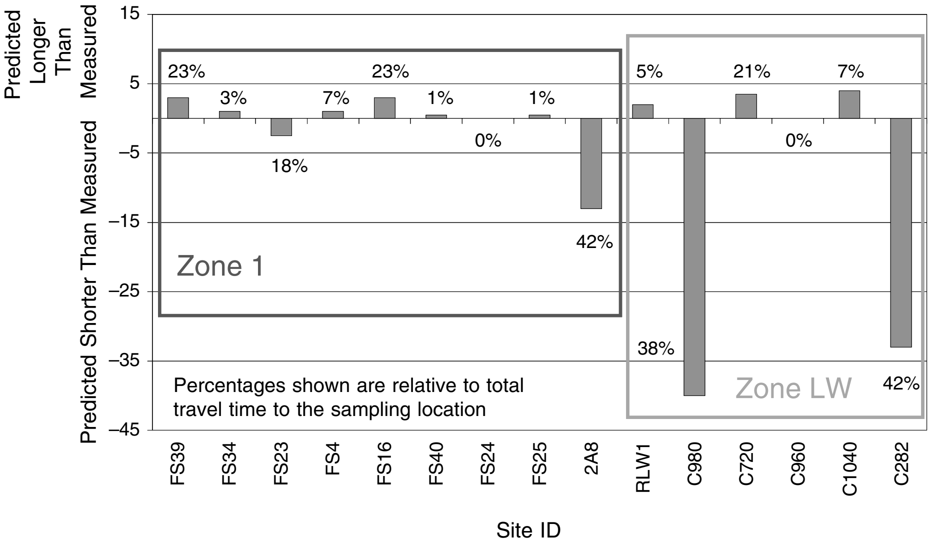

Figure 14 Difference in hours between travel time predicted by hydraulic model and observed during the tracer study

appropriate operational data, including plant flows and chemical dosages. The predicted DBP data generated during the simulation were compared with the results of the SDS and the field results. Correction factors were then calculated and written into the WTP model code such that the model accurately predicted DBP formation in the distribution system.

SUMMARY AND CONCLUSIONS

The tracer testing program provided the travel time and source contribution data necessary for model calibration. The following conclusions can be made from data collected during the tracer test period:

- Thirteen of the 17 sampling points (originally selected from “high TTHM” data) had travel times of less than three days; however, there were locations at the extremities of the system that experienced travel times of five to eight days.
- City Site 750 is at a dead end in the system and experienced the longest travel time, highest TTHMs, and lowest chlorine residual of all locations sampled. The City is aware of these conditions

and routinely flushes this location (approximately every two weeks), but did not do so during the study.

- Source contribution from Verde Wells was calculated roughly as 10% at most locations in Zones 1, LW, and 3S based on fluoride breakthrough results. However, at site Booster 2A8, the contribution was much more significant.
- Under the conditions of this study, SDS tests were good predictors of DBP formation in the distribution system.
- Chlorine residual was a good indicator of system DBP concentrations.
- While long detention times (up to eight days) were observed in the system, the rate of DBP formation appeared to slow dramatically after 72 hr under the conditions of this tracer test. For the VVWTP water, TTHMs in the distribution system were 10 µg/L (about 16%) higher at 206 hr than they were at 72 hr. However, this difference can be expected to be greater in the summer due to higher temperatures.
- The hydraulic model accurately accounts for the influence of Verde Well water in Zone 1 and Zone LW.
- Calibration of the hydraulic model resulted in these differences between the observed and computed travel times:
  - Less than or equal to 5% at 6 of the 15 sites
  - Less than or equal to 10% at 8 of the 15 sites
  - Less than or equal to 25% at 12 of the 15 sites
  - Less than or equal to 50% at all of the 15 sites
- The largest differences between the observed and modeled fluoride response curves in the calibration analysis may be due to difficulties encountered when flushing large-diameter sample collection lines. That is, the distribution line may not have been adequately flushed, resulting in gradual and delayed dropoffs in the observed fluoride concentrations.

ACKNOWLEDGMENTS

The authors would like to thank the following organizations for their help in organizing and implementing the tracer study: City of Phoenix Water Services Department, City of Phoenix Fire Department, and Narasimhan Consulting Services.

CHAPTER · 51

Modeling Water Quality in a Multisource Network

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SUMMARY

The accuracy of distribution system water quality models for reactive substances such as chlorine is dependent on how the reaction processes are modeled, the hydraulic flow model, and the quality and quantity of field data available for model calibration. The current trend in modeling reaction processes is to separate bulk and wall reactions. However, current water quality models still treat bulk reaction coefficients as pipe properties. The limitation of this approach in multisource networks is that the bulk reaction coefficient associated with a given pipe remains constant with time irrespective of changes in the blend of water passing through the pipe.

A more appropriate approach in multisource networks may be to treat the bulk reaction coefficient as a source water property and to treat the bulk reaction coefficient in a given pipe as a variable dependent on the blend of water passing through the pipe. This approach requires an appropriate method for computing bulk reaction coefficients for different blends of source waters given the bulk reaction coefficients of the sources.

This chapter presents and investigates an approach to modeling and calibrating water quality models that accounts for the variation of bulk reaction coefficients within pipes due to in-network source water blending. This approach is then applied to model water quality in one of the service areas of South Central Connecticut Regional Water Authority receiving water from groundwater and surface water sources.

INTRODUCTION

Traditionally, water quality models lump bulk and wall reactions into a reaction model described by a single reaction coefficient [1]. The reaction coefficients are then used as calibration parameters and are treated as pipe and tank attributes. More recently, water quality models have begun to separate bulk and wall reactions [2]. This requires reaction coefficients to describe the bulk reactions as well as the wall reactions. The wall reaction coefficients depend on the characteristics of the pipes, among other factors, and are therefore treated as pipe attributes. The bulk reaction coefficients depend on the characteristics of the water and are independent of pipe characteristics. Separation of bulk and wall reactions allows one to measure the bulk reaction coefficients in a laboratory and treat the wall reaction coefficients as calibration parameters. However, most water quality models still assign the measured bulk reaction coefficients to pipes and treat them as a pipe attribute that remains constant for any given pipe during a simulation.

Treating bulk reaction coefficients as pipe attributes seems reasonable when the system is fed by a single source of water or by two or more sources of water of similar reaction characteristics. However, this approach can be restrictive when modeling a system that is fed by two or more sources of water of widely different reaction characteristics. In such cases, the bulk reaction coefficient of water within any given pipe is a dynamic quantity. As the source contribution of the different source of water within a given pipe changes with time, so will the bulk reaction coefficient within that pipe.

This chapter suggests and investigates an alternative approach to modeling bulk reaction coefficients in water quality models. With this approach, the bulk reaction coefficients are treated as attributes of the sources of water instead of the pipes. Such an approach cannot only simplify the water quality calibration process, but also reduce the amount of data needed for water quality models.

MODELING APPROACH

For the purpose of this study, it will be assumed that bulk reactions can be described by first-order kinetics according to the following equation.

$$\frac{dC}{dt} = -k_b C \qquad (1)$$

Here C is the concentration of the substance being modeled and k_b is the first-order bulk reaction coefficient. The wall reactions will be assumed to follow first-order kinetics as well according to the following equation.

$$\left.\frac{dC}{dt}\right|_{wall} = -\frac{k_{w1}C_w}{r_h} \tag{2}$$

Here C_w is the substance concentration at the wall, r_h is the hydraulic radius of the pipe, and k_{w1} is the first-order wall reaction coefficient.

Consider the two-pipe system shown in Figure 1. The two pipes are identical in every respect. The system is supplied by source S_1 at a constant rate. Source S_1 is chlorinated and has a chlorine concentration of C_0. Let the bulk reaction coefficient of chlorine in the source water from S_1 be k_{high}. Also, let us assume that there is no wall reaction in this two-pipe system.

If the system is allowed to operate in this mode for a sufficient time, the concentration of chlorine in the system will assume a steady state. When the system reaches steady state, the concentration profile of chlorine in the system will appear as shown by the solid line in Figure 2.

Here, C_{b1} is the steady-state concentration at node B, and C_{c1} is the steady-state concentration at node C. Let us assume that at some point in time, source S_1 stops flowing and the system is now supplied by source S_2 injected at node C, as shown in Figure 3. Source water S_2 is also chlorinated to a concentration of C_0. However, the bulk reaction coefficient of chlorine in source water S_2 is k_{low} where k_{low} is much smaller than k_{high}.

If the system is allowed to operate in this manner, the chlorine concentration will again reach a steady-state distribution in the system, as shown by the dashed line in Figure 2. The steady-state concentration at node B is now C_{b2}, which is higher than C_{b1}. As the system moves from

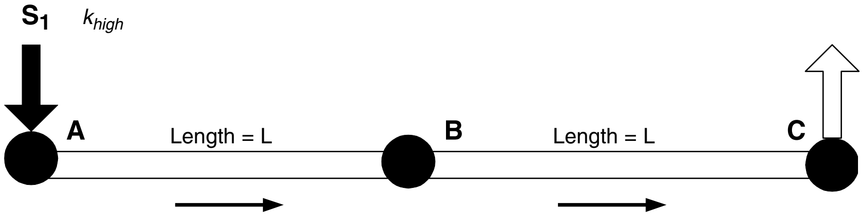


Figure 1 A simple two-pipe system

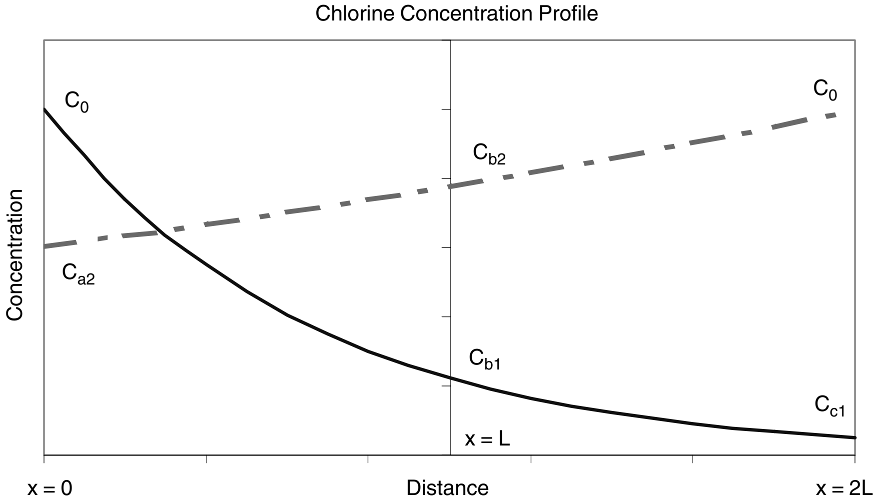


Figure 2 Steady-state concentration profile in the two-pipe system

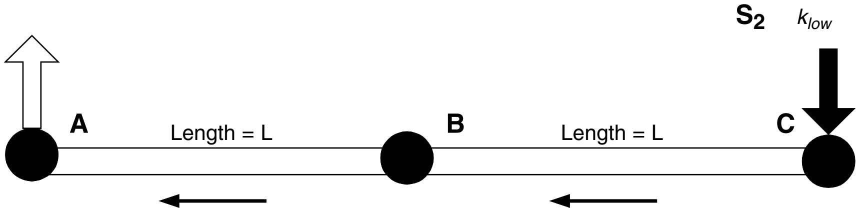


Figure 3 Source of water changes in the two-pipe system

the first steady state to the second steady state, the concentration at nodes A, B, and C will vary with respect to time.

The variation of concentration at node A is shown in Figure 4A. Note that it is possible for the chlorine concentration at node A to fall below C_{a2} before it reaches a steady-state value of C_{a2}. This is because when source S_2 starts to flow, the water residing in the system is from source S_1. As the water from source S_1 is flushed out of the system, chlorine in this water will continue to react with a bulk reaction coefficient of k_{high}. Similarly, the variations in concentration at node B and node C are shown in Figure 4B and Figure 4C respectively.

If one were to use a water quality model to predict the chlorine concentrations at the three nodes, what bulk reaction coefficients should

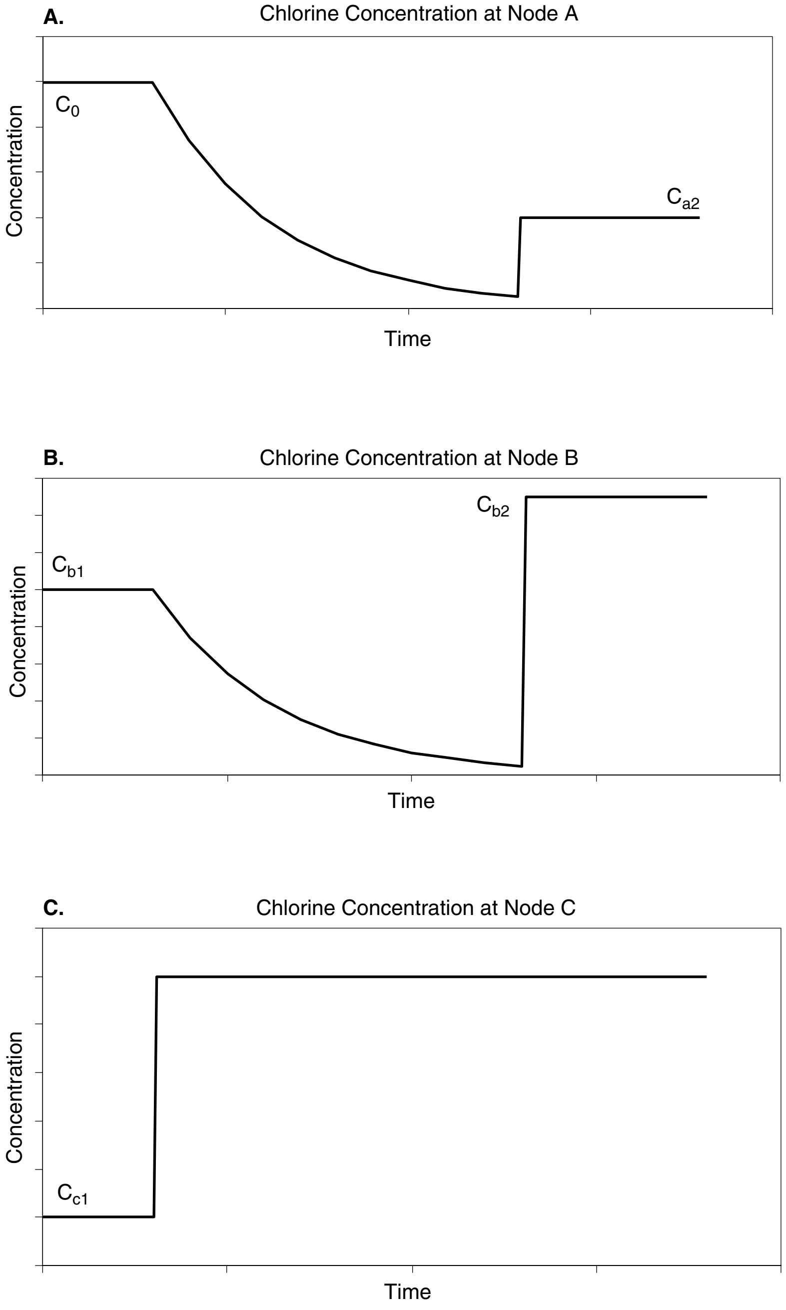


Figure 4 Variation of chlorine concentration at node locations

one assign to the two pipes? Should it be k_{high} or k_{low}? One possibility is to use bulk reaction coefficients assigned to the two pipes as calibration parameters and vary them between k_{high} and k_{low}. However, even this can be a difficult task. If we assign fixed reaction coefficients to the two pipes, then it can be shown that for first-order reaction kinetics this would imply that C_{c1} is equal to C_{a2}. However, C_{c1} and C_{a2} can differ considerably in reality. Depending on the difference between the observed values of C_{c1} and C_{a2}, calibrating the water quality model by assigning fixed reaction coefficients to pipes can be a frustrating exercise even for this simple two-pipe system.

There seems to be good intuitive and practical reason, then, to treat bulk reaction coefficients as properties of the source water instead of the pipes. This approach mimics reality and can simplify the calibration of water quality models. It can also reduce the amount of data needed for water quality modeling since this approach does not require a bulk reaction coefficient for every pipe in the system. Treating bulk reaction coefficients as source water properties does add some computational requirements to the model. As parcels of water from different sources enter the system, they have to be assigned bulk reaction coefficients. These bulk reaction coefficients are "carried" by the parcels of water as they move through the system (Figure 5). The chlorine within a parcel of water decays according to the bulk reaction coefficient assigned to that parcel.

As a parcel of water travels through a pipe, its bulk reaction coefficient remains the same as it was when it entered the pipe. However, when that parcel of water reaches the downstream end of the pipe, it is possible that this parcel will mix with other parcels of water that have their own bulk reaction coefficients. What should be the bulk reaction coefficient of the mixture of the two parcels? This is a difficult question that needs research. In this study, a simple approach was taken. When

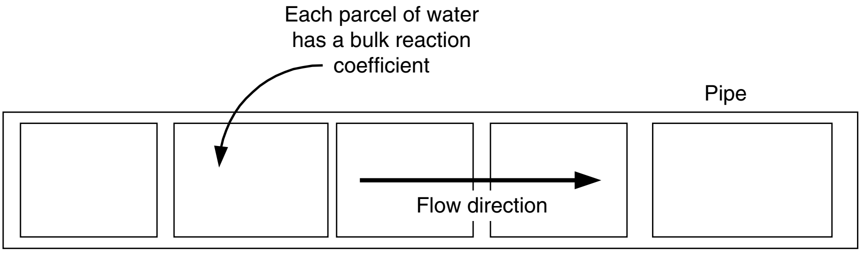


Figure 5 Water parcels must "carry" their bulk reaction coefficients

two parcels of volume V_1 and V_2 and bulk reaction coefficient k_{b1} and k_{b2} mixed, the bulk reaction coefficient of the mixture, $k_{b,mix}$, was computed using a volumetric average.

$$k_{b,\,mix} = \frac{V_1 k_{b1} + V_2 k_{b2}}{V_1 + V_2} \tag{3}$$

Tanks were assumed to be completely mixed and an equation like that above was used to calculate the changes in bulk reaction coefficients within tanks. When a parcel of water of volume V_1 with a bulk reaction coefficient of k_{b1} entered a tank with a water volume of V_T and a bulk reaction coefficient of k_{bT}, the resultant bulk reaction coefficient in the tank, $k_{b,mixT}$, was computed using the following equation.

$$k_{b,\,mixT} = \frac{V_1 k_{b1} + V_T k_{bT}}{V_1 + V_T} \tag{4}$$

The following sections describe the application of this modeling and calibration approach to a real water distribution system. The distribution system selected for this example application is the Whitney-Wintergreen service area of South Central Connecticut Regional Water Authority. This service area was selected because it receives water from both surface water and groundwater sources, which typically have different reaction characteristics.

DESCRIPTION OF THE WHITNEY-WINTERGREEN SERVICE AREA

South Central Connecticut Regional Water Authority serves a population of approximately 400,000 in 12 towns in south-central Connecticut. Whitney-Wintergreen is one of the service areas served by the Regional Water Authority. This service area serves approximately 13,000 customers and is represented in a hydraulic model by 1,100 nodes and 1,500 pipes (Figure 6). The Regional Water Authority's hydraulic model includes every pipe in the service area. Demands were assigned to nodes using a geographical information system program and actual annual billing data. Piping is a mix of ductile iron and cast iron; about 70% of the pipe interiors are cement-lined. Pipe Hazen Williams factors were universally set to 100 in the model. The service area contains a single 1.5-million-gallon

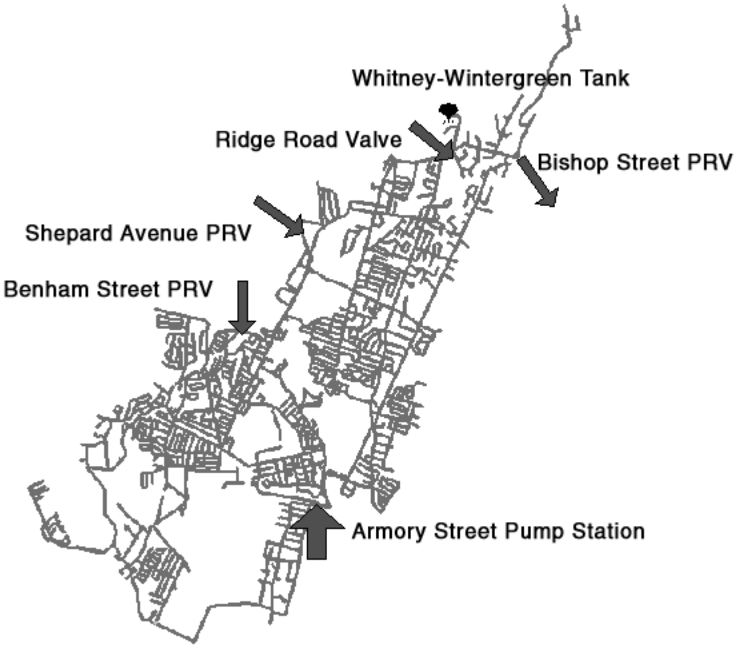


Figure 6 Whitney-Wintergreen service area

Whitney-Wintergreen storage tank. The service area is served primarily by surface water supply from New Haven Service Area in the south, through the Armory Street Pump Station. It also receives supplemental groundwater-based supply from the York Hill Service Area in the north through Benham Street and Shepard Avenue PRV. As supplemental sources, except during the summer, the flow through these PRVs is normally very low relative to the flow from Armory Street, and frequently is zero. The Ridge Road throttling valve also provides supplemental supply from York Hill to Whitney-Wintergreen. Its purpose is to aid in filling the Whitney-Wintergreen tank and it is not generally kept open or closed all day long. Bishop Street PRV sends water from Whitney-Wintergreen Service Area to the Reduced Rabbit Rock Service Area. This is a backup source for Reduced Rabbit Rock Service Area, so Bishop Street PRV is infrequently open. In 2001, average daily demand in the service area was 5.1 million gallons per day.

METHODOLOGY

This section describes the general methodology used to calibrate the Whitney-Wintergreen Service Area model hydraulically and for water quality calculations. There were several sources of uncertainty in this calibration exercise. Therefore, the results of this calibration exercise should be considered only qualitative in nature and for illustrative purposes only. Further refinement of hydraulic and water quality calibration is ongoing.

The Whitney-Wintergreen model was hydraulically calibrated using hourly SCADA data from May 12, 2001. The SCADA data consisted of pressures and flows at all the boundary points of Whitney-Wintergreen: Benham Street PRV, Shepard Avenue PRV, Ridge Road Valve, and Armory Street Pump Station. In addition, tank levels in the Whitney-Wintergreen tank were available.

In addition to the hydraulic information, water quality information was collected at specific locations in the system. Water samples were taken on May 10, 2001, at the four boundary points where water can enter the service area. These water samples were used to measure first-order bulk reaction coefficients of free and total chlorine. In addition, free and total chlorine concentrations were monitored in the system at specific locations over a period extending from May 9 to May 11, 2001.

In order to calibrate the model hydraulically, the flows obtained from SCADA at the entry and exit points of the system were imposed on the model as boundary conditions. The SCADA tank levels in the Whitney-Wintergreen tank were used to estimate the tank inflow or outflow. Using the SCADA flows at the entry and exit points of the system and the estimated tank flows, a flow balance was done to estimate the demand variation over the 24-hr calibration period. The calibration process included adjusting demands in the model to match the estimated demands and adjusting friction factors.

Once the model was hydraulically calibrated, water quality calibration was initiated. Free chlorine concentrations monitored over a 24-hr period on May 10, 2001, were used for water quality calibration. The free chlorine concentrations measured at the entry points of the system were imposed on the model as boundary conditions. The bulk reaction coefficients obtained from water samples at the entry points of the system were also imposed on the model. The wall reaction coefficient was then used as a calibration parameter to match the measured free chlorine concentrations at some of the points in the interior of the system.

RESULTS

The accuracy of hydraulic calibration was assessed by comparing measured with predicted pressures at locations where SCADA information on pressures was available. Comparison of measured and predicted pressures for two of the points is shown in Figure 7. The predicted pressures at all the SCADA points were within 5–10 psig of the measured pressures except at 4 hr, when the difference between predicted and measured pressures was between 10 and 18 psig at some of the SCADA points. No measured flow information was available at any points in the interior of the service area.

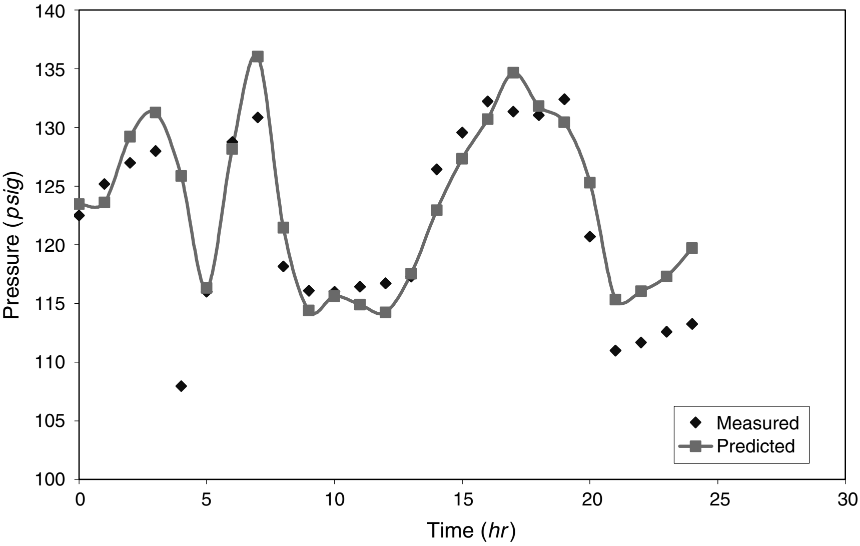
First-order bulk reaction coefficients were determined for free chlorine and total chlorine for water samples taken at all the entry points of the Whitney-Wintergreen system using standard bottle tests. The following table shows the first-order bulk reaction coefficients for free chlorine.

Location	Bulk Reaction Coefficient (*1/day*)
Benham Street PRV	0.02
Shepard Avenue PRV	0.01
Ridge Road Valve	0.02
Armory Street Pump Station	0.05

These bulk reaction coefficients were imposed on the model. Bulk reaction coefficients were not used as calibration parameters. The boundary conditions for water quality simulation were obtained from measured free chlorine concentrations at all the entry points of the service area. An estimate was needed for the distribution of free chlorine in the system at the start of the simulation. This estimate was obtained by first assuming a steady-state distribution of free chlorine in the system. A 24-hr simulation was then run. Let us call this Run 1. A 24-hr simulation was then run again, but this time the final concentrations from Run 1 were used as starting concentrations. Let us call this Run 2. The final concentrations from Run 2 were then used as initial conditions for Run 3. This process was repeated a few times. The final concentrations from the last run were then used as initial conditions for the final simulation.

At first, a 24-hr water quality simulation was run assuming that there was no wall reaction. The free chlorine concentrations predicted by the model at some of the interior points of the service area were higher than the measured free chlorine concentrations at these points. This suggested a possibility that wall reaction could also be going on in the

A. Armory Street Pump Station



B. Skiff Street and State Street

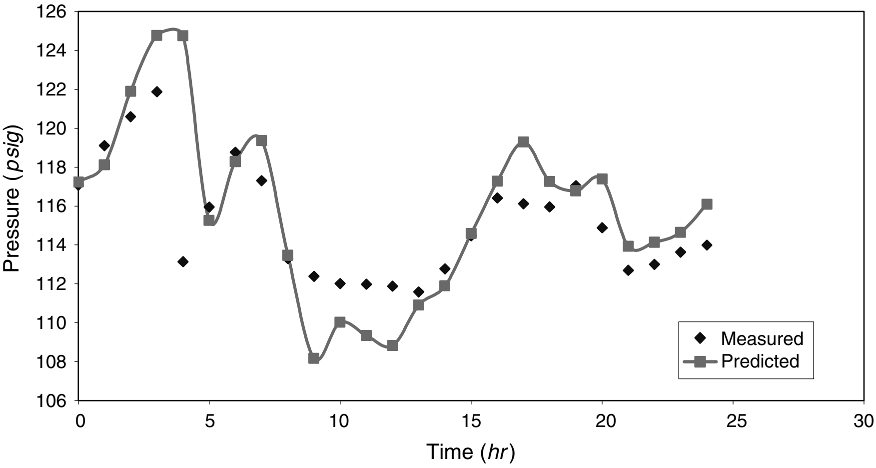


Figure 7 Comparison of measured and predicted discharge pressures at different pump stations

system. Wall reaction coefficient was then used as a calibration parameter. Wall reaction was assumed to be of first order and a value of 0.1 ft/day was found to provide a reasonable fit between the predicted and measured concentrations. This wall reaction coefficient was applied to all the pipes.

Figure 8 shows the comparison of measured and predicted free chlorine concentrations at three locations in the interior of the service area.

Since bulk reaction coefficient was treated as a property of source water and the bulk reaction coefficients of different water parcels were tracked through the system, it is of interest to see the distribution of bulk reaction coefficient within the system. Figure 9 shows the distribution of bulk reaction coefficient in the system at the end of the simulation.

With this modeling approach, the bulk reaction coefficient of water within a given pipe is not necessarily constant. Figure 10 shows the simulated variation of bulk reaction coefficient within one of the pipes in the service area. This pipe receives water from both the Armory Street Pump Station and the Shepard Avenue PRV. Shepard Avenue PRV flow is intermittent during the calibration period. During the time when the Shepard Avenue PRV is flowing, the bulk reaction coefficient in this pipe declines.

CONCLUSIONS

This chapter has presented an alternative approach to modeling bulk reaction coefficients in water quality models. The traditional approach for calibrating water quality models treats bulk reaction coefficients as pipe attributes. However, it makes more intuitive and practical sense to treat bulk reaction coefficients as source water properties. This can simplify the calibration of water quality models and also reduce the amount of data required.

A water quality model of the Whitney-Wintergreen Service Area was calibrated using the approach suggested in this chapter, and reasonable results were obtained. These results were presented for illustrative purposes only. There were several sources of uncertainty in this calibration exercise. A limited amount of information was available for hydraulic calibration; the hydraulic calibration was based on SCADA data from May 12, 2001; and the water quality data used for calibration were from May 10, 2001. Also, it was assumed that a volume averaging approach could adequately describe the bulk reaction coefficient of a mixture of two or more different kinds of sources. The hydraulic and water quality calibration for this service area is ongoing and needs more refinement. However, the preliminary results indicate that the modeling and calibration approach described in this chapter is practical and can give reasonable results.

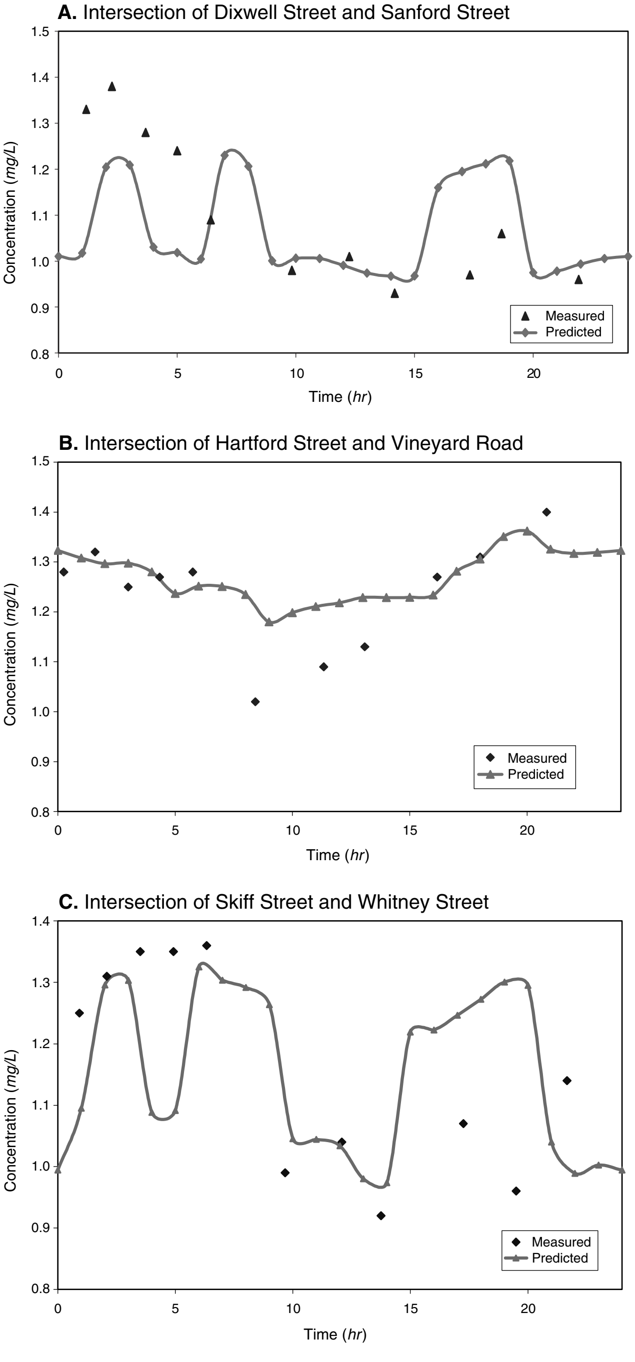
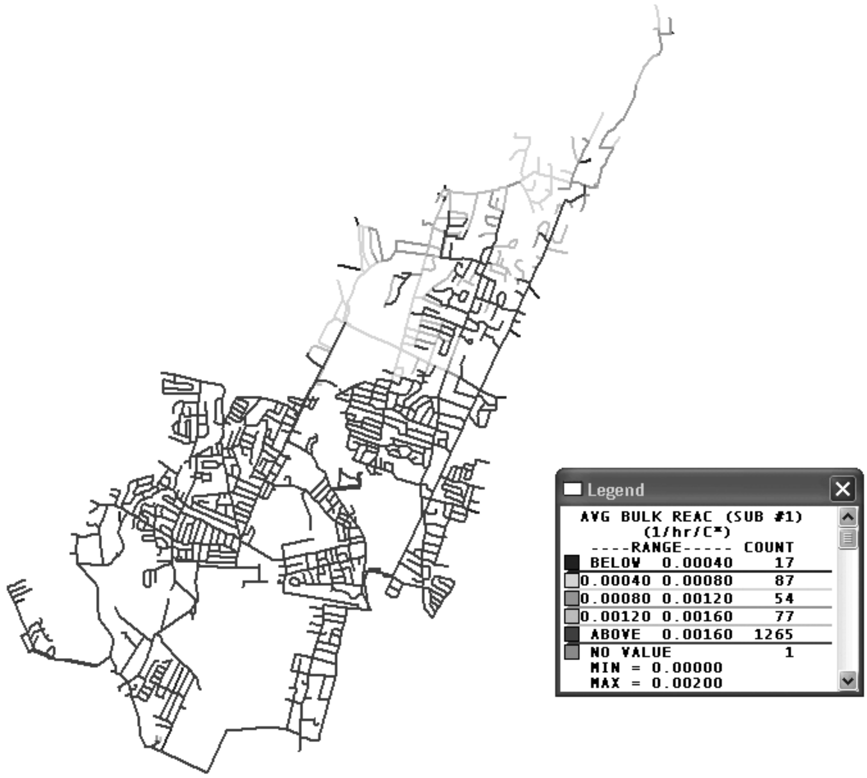


Figure 8 Comparison of measured and predicted free chlorine concentrations at various locations



```
  AVG BULK REAC (SUB #1)
        (1/hr/C*)
   ----RANGE-----  COUNT
   BELOW  0.00040     17
 0.00040 0.00080     87
 0.00080 0.00120     54
 0.00120 0.00160     77
   ABOVE  0.00160   1265
   NO VALUE            1
   MIN = 0.00000
   MAX = 0.00200
```

Figure 9 Distribution of bulk reaction coefficient within the system at 24 hr

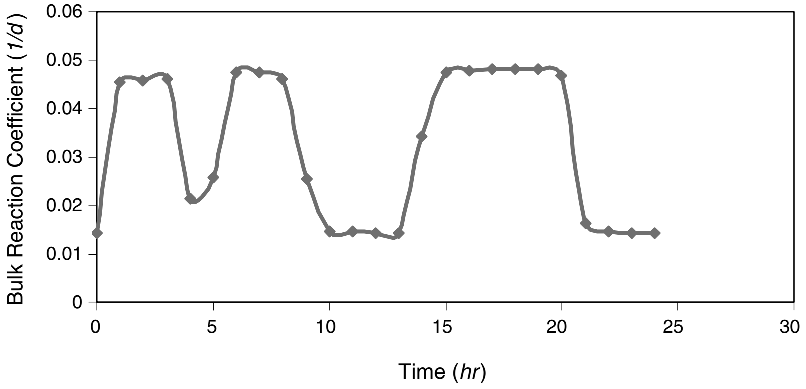


Figure 10 Variation of bulk reaction coefficient within one of the pipes

ACKNOWLEDGMENT

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CHAPTER · 52

Water Quality Modeling of a Complete Distribution System: Development, Results, and Practical Applications

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BACKGROUND

Recognizing the increased regulatory emphasis on water quality in the distribution system, the Cincinnati Water Works (CWW) is implementing a dynamic water quality model for its distribution system. The long-range goal is to use the water quality model as an important tool in evaluating and addressing a wide range of water quality issues and continued regulatory compliance associated with the distribution system water quality. This chapter presents CWW's effort associated with the development, implementation, and potential applications of the water quality model.

PROJECT OBJECTIVES

The water quality modeling project started in January 2000 and is scheduled to be completed in the summer of 2001. CWW's existing hydraulic and extended period simulation models, which include approximately 63% of the 2,750 miles of water mains in the distribution system, form the basis for the water quality model. The objectives of the water quality model are as follows:

- To build a dynamic water quality model upon the foundation of the extended period simulation portion of CWW's existing hydraulic model
- To achieve a calibration goal of ±0.1 mg/L between the model predicted chlorine residual and actual field measurements

- To develop and prioritize water quality model applications that will assist CWW with further understanding and optimizing water quality in the distribution system

CWW SYSTEM OVERVIEW

The CWW system consists of two water treatment plants and about 2,750 miles of water mains. The largest treatment plant is the Richard Miller Treatment plant with a rated capacity of 220 mgd and an average-day water production of about 113 mgd. The plant serves about 88% of CWW's customers and uses the Ohio River as its raw water source. The plant uses conventional treatment consisting of rapid mix, flocculation, sedimentation, filtration, and postfilter granular activated carbon. CWW's second plant is the Charles M. Bolton Treatment plant with a design capacity of 40 mgd and an average day flow of about 15 mgd. This plant uses a groundwater source and serves about 12% of CWW's customers. Both plants use chlorine as a primary disinfectant and feed into a common distribution system with a floating interface between finished water sources. CWW's distribution system consists of eight pressure zones, 14 pump stations, and 19 finished water storage tanks. The total storage capacity is 155 MG, and storage tank sizes range from 0.5 to 80 MG. Most storage tank sizes, however, range from 1 to 2 MG. Seven storage facilities are equipped with continuous chlorine residual analyzers. Approximately 40% of the water mains in CWW's system are unlined cast-iron water mains.

HYDRAULIC MODEL BACKGROUND

The existing hydraulic and extended period simulation (EPS) models form the basis for the water quality model. These models were developed using CWW's GIS database, which contains information on the diameters and lengths of all water mains within the CWW distribution system. The primary purpose of the hydraulic model was to develop a long-range distribution system improvement plan, and consequently it was neither practical nor necessary to include every pipe in the system in the model. Therefore, prior to initiating the hydraulic analyses, the distribution network was skeletonized by deleting some of the smaller-diameter pipes from the model database. The skeletonized model still contains about 1,700 miles of pipe, or about 63% of the total pipe length within the CWW system. In general, the skeletonized hydraulic model contains all of the 12-in. and larger mains, about 85% of the 10-in. and 8-in. mains, and about 25% of the 6-in. mains. It is worth noting, however, that the hydraulic model contains over 90% of the total volume within the distribution system pipes.

The EPS model is an enhancement to the static model and provides for a dynamic analysis of the distribution system that simulates varying operations over a period of time. EPS models include diurnal demand patterns and operational control statements, and therefore these types of models are very useful for distribution system operational studies and water quality analyses.

It was recognized that a successful water quality model requires accurate prediction of hydraulic parameters throughout the distribution. In CWW's hydraulic model, the difference between the measured and predicted flow is typically less than 7%, reservoir level differences between the model and the field are typically less than 1.2 ft, and the pressures are within ±5 psi. Considering the size of CWW's distribution system, these calibration parameters were considered acceptable for proceeding with the water quality modeling.

CHLORINE MODELING PILOT TEST

Prior to initiating a systemwide water quality model, CWW conducted a pilot-area water quality model. The purpose of the pilot test was to generally assess the model's capabilities for simulating and predicting chlorine residual concentrations in an old distribution system with primarily unlined cast-iron pipes. The pilot study also provided the opportunity to determine data needs and sampling requirements to support the calibration of a systemwide water quality model. The pilot area consisted of an isolated area of the distribution system with primarily unlined cast-iron pipes, some of which date back to 1884. All pipes in this pilot area were included in the model.

A 24-hr field-testing program for flow and chlorine residual was developed for the pilot test area. The testing program involved chlorine residual measurements at 21 fire hydrants at 4-hr intervals over a 24-hr period. In addition, detailed flow measurements were collected to develop a specific diurnal curve for the test area. Chlorine can be lost due to both reactions with available organic matter in the bulk water and interaction with the pipe wall material. The bulk water chlorine decay rate was measured to be 0.22/day for the month the pilot test was performed. To estimate the pipe wall decay coefficients for the pilot area, specific flow and chlorine residual sampling focused on four pipe segments. These four segments were unlined cast-iron pipes that were considered representative of the conditions in the pilot area. It was concluded from these tests that the pipe wall reactions, as opposed to the bulk water reactions, were responsible for 95% of the total chlorine decay in the area. The results for six different tests on each pipe segment, however, showed considerable variation in the computed wall decay coefficients for each pipe, as shown in Table 1. The mean wall decay coefficient ranged from 3.8/day to 10/day. The average pipe wall decay coefficient for the four segments was 6.22/day.

A comparison of the chlorine residual results predicted by the model and the field measurements is shown in Figure 1. The difference between the field and modeled data at the 21 sample sites ranged from 0.08 to −0.20 mg/L. The average difference between the 21 sites was 0.01 mg/L, with a standard deviation of ±0.073 mg/L.

A comparison between the instantaneous field test results and corresponding model results showed that 13 of the 21 locations had modeled chlorine residual concentrations within 0.1 mg/L of the field data, and 18 of the sample sites were within 0.15 mg/L. The highest difference measured, −0.26 mg/L, was on a dead-end main.

The key findings obtained from the pilot test area were as follows:

- It was possible to achieve a reasonably close calibration for chlorine residual. Comparing modeled versus 24-hr average results, the final range of differences was +0.09 to −0.11 mg/L. Comparing instantaneous samples, differences ranged from +0.12 to −0.22 mg/L.
- For the pilot area, a calibration goal of ±0.1 mg/L appeared reasonable for the 24-hr average results, while a calibration goal of ±0.2 mg/L appeared reasonable for instantaneous values.
- About 95% of the chlorine decay was attributable to the unlined cast-iron pipe walls.

WATER QUALITY MODEL SAMPLING PLAN

Based on the successful results of the water quality of the pilot study area, CWW continued with the implementation of the water quality model that encompassed the majority of the distribution system. The existing hydraulic model, which includes about 63% of the 2,750 miles of the water mains in the system, was used as the basis for the water quality model. No additional mains were added to the hydraulic model for water

Table 1 Pipe wall decay coefficient test results

Location	Diameter (*in.*)	Year Installed	Min. Wall Decay (*1/day*)	Max. Wall Decay (*1/day*)	Avg. Wall Decay (*1/day*)
1	6	1908–1910	5.78	11.95	10.03
2	6	1888–1892	5.39	8.01	6.70
3	10	1909	0.78	9.60	4.31
4	6	1927	0.15	8.15	3.82

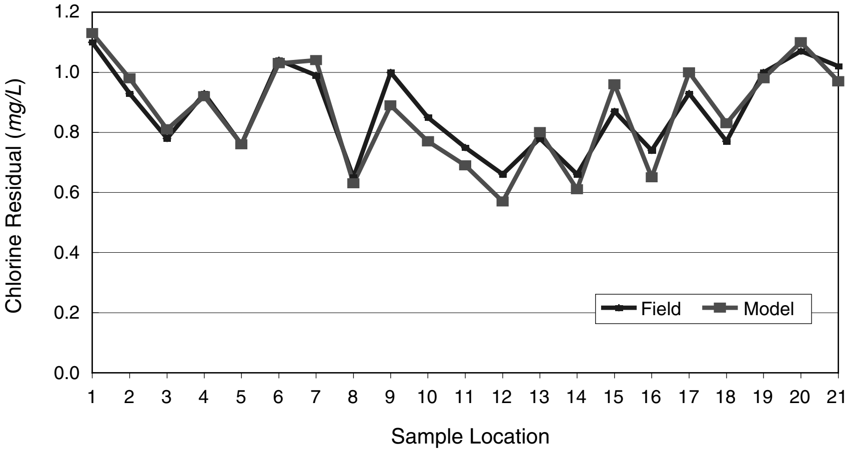

Figure 1 Field and modeled chlorine residual concentrations

quality modeling purposes. However, the hydraulic model's ability to accurately predict flow, pressure, and tank levels throughout the distribution system was reviewed prior to starting the water quality modeling.

A sampling plan of the distribution system was developed to collect sufficient data for calibrating the water quality model. Due to the size of CWW's system, it was necessary to divide the system into nine separate sampling areas. This allowed for sampling of each area sequentially for one week over a nine-week period. Generally, the sampling areas were consistent with CWW's pressure zones, although some smaller pressure zones were combined, while some of the larger zones were subdivided into smaller areas. Also, the sampling areas were generally equal in size such that the number of samples and the staffing needs in each area would be reasonably consistent over the course of the sampling. The sampling plan consisted of two key activities: (1) continuous chlorine residual monitoring, and (2) grab sampling over a 24-hr period. The following sections discuss these sampling activities in greater detail.

Continuous Chlorine Residual Monitoring

The objective of the continuous monitoring was to collect chlorine residual data continuously over a one-week period on the primary water mains feeding into a specific sampling area. This would provide the model developers with detailed information on the variability of the chlorine residual entering the sampling area prior to the actual grab sampling event. Typically, chlorine monitors were installed at preselected locations on Monday and the 24-hour grab sampling was started on Thursday evening. In all, 40 different monitoring sites were selected, with an average of four monitors per sampling area.

The equipment for the continuous monitoring task had to be very flexible and durable because CWW did not own aboveground facilities at each of the selected monitoring sites. Therefore, the monitoring equipment needed to have the following capabilities:

- Built-in data logging
- Battery power
- Operable in valve vaults that are prone to flooding
- Internally adjust for pH without the use of reagents
- Unaffected by flow and pressure variations
- Measure chlorine residual within ± 0.1 mg/L accuracy
- No sample withdrawal or discharge

CWW evaluated various monitors and ultimately procured six monitors made in Great Britain that met all the capabilities listed above. Generally, the chlorine residual monitor selected by CWW is an insertion-type probe that measures chlorine residual directly in the water as it travels through the main. The probe is inserted into the main through a 2-in. corporation stop and the unit is provided with a stand-alone battery and data logger. Because at the time of this project the monitors were not yet manufactured specifically for the US market, CWW had to manufacture special adapters to install the monitor onto US-type corporation stops. While this required some research into the design of British pipe threads, it allowed CWW to use many of its existing corporation stops, which ultimately reduced the anticipated installation time. Nevertheless, the process of locating and field verifying up to 40 different monitoring sites proved to be time consuming. The continuous monitors collected data every five minutes over a 24-hr period for seven days. Ultimately, CWW anticipates further use of these types of monitors in focused areas to characterize specific variations in water quality.

Grab Sampling Plan

In addition to the detailed information collected by the continuous monitors, grab samples were collected from fire hydrants at 4-hr intervals over a 24-hr period. Each sample area contained approximately 25 sample collection sites, such that about 150 data points were collected for each sample area over a 24-hr period. For the nine different sample areas in the distribution, about 200 different grab sample sites were selected that generated 1,200 data points for the calibration of the water quality model. All samples were collected and analyzed in the field. Duplicate samples were collected at each sample site to verify the results. In addition to free chlorine residual, pH and temperature were collected at least once at each sampling hydrant. Also, up to 10 THM, TOC, and HAA samples were collected in each sampling area to develop disinfection by-product data as a function of water age.

Grab Sample Site-Selection Criteria

All 200 grab sample sites were carefully selected in order to provide spatial coverage of the distribution system and to adequately capture the unique physical characteristics of CWW's distribution system. Specific criteria that were considered in the grab sample site selection included the following:

- Pipe diameter
- Pipe material of construction
- C-value
- Water age
- Areas with chlorine residual between 0.2 and 0.5 mg/L

The pipe diameter criterion focused on locating the sample sites on pipe sizes consistent with the pipe sizes in the distribution system, and with the pipe sizes in the model. In other words, the sample sites should not favor certain pipe sizes, but rather reflect the characteristics of the distribution system. Table 2 illustrates the pipe sizes in CWW's distribution system and the percentage of the pipe sizes included in CWW's model.

CWW's distribution system contains approximately 2,750 miles of pipe. The model includes about 63% (1,756 miles) of the total miles of pipe in the system. As was previously discussed, the model includes all pipes larger than 12 in., 85% of the 8-in. to 10-in. pipes, and about 25% of the 6-in.-diameter pipes. It was CWW's objective that the grab sample sites be representative of the distribution system, but this objective was limited by the actual pipe sizes included in the model. Table 3 shows the percentage of the final selected grab samples relative to the percentage of pipe length in the model.

As shown in Table 3, the grab sample distribution was representative of the piping distribution in the model, except for the 8-, 10-, and 12-in. pipes. This was partially due to maintaining spatial coverage of the entire system and compliance with the other sampling selection criteria listed earlier. However, 82% of the samples were collected from pipe sizes of 12 in. or less, while this group of pipe sizes represents 80% of the pipes in the model. Considering the other sampling criteria, this distribution concentrated adequately on the smaller pipe sizes, which would be favorable from a water quality standpoint.

The pipe material of construction was another important criterion. Unlined cast-iron pipe could have a chlorine decay due to the pipe wall up to 10 times greater than cement-lined pipe.

CWW's distribution system consists of approximately 40% of unlined cast-iron pipe. With the aid of the GIS system, about 41% of the grab sample sites were located on unlined cast-iron pipes. In addition, the grab sample sites in each sampling area were compared with the percentage of the unlined cast-iron pipe lengths in that area. This was to ensure that the sample sites on unlined cast-iron pipes were adequately represented in each sampling area as well as for the entire distribution system.

The C-value was also considered in the sample site selection. Typically, the decay of chlorine due to wall effects increases as the C-value decreases. CWW's calibrated hydraulic model of the distribution system includes C-values assigned to each pipe. This information was generally used to avoid pipes with C-values below 50 and above 140. Areas with extremely low C-values would be addressed separately in the future in a more focused application of the water quality model.

The water age values predicted by CWW's EPS models were reviewed to ensure that the grab samples included the entire range of potential water age in CWW's system. Special emphasis was placed on areas with extremely high water age to ensure that sufficient data were collected in these areas.

Table 2 CWW's distribution system and model pipe sizes

Pipe Diameter (*in.*)	Pipes in Actual System (%)	Pipes in Model (%)	Length of Pipes in Model (*miles*)
6	40.0	25	273
8–10	37.4	85	871
12	10.3	100	285
16	3.6	100	98
20	0.8	100	22
>24	8.0	100	207

Table 3 Grab sample site distribution

Pipe Diameter (*in.*)	Normalized Pipes in Model (%)	Grab Samples (*% of total*)
6	15	14
8–10	49	33
12	16	35
16	6	10
20	1	0
>24	12	8

Areas in the distribution system that have experienced chlorine residual between 0.2 and 0.5 mg/L in the past four years were also included in the grab sampling plan. CWW has approximately 90 routine sampling points in the distribution system. While routine sampling point results are in compliance with Ohio EPA (OEPA) regulations for chlorine residual, certain areas were included in the grab sampling plan to better characterize the chlorine residual over a 24-hr period.

Grab Sampling Implementation

The entire sample site selection process described above was performed in the office with various tools such as GIS, hydraulic models, and historical water quality data. Even specific hydrant sampling locations could be selected in the office with a GIS system. However, it was critical to perform final field verification of each sampling hydrant before the actual sampling could proceed. This was necessary to ensure that the selected locations would be practical with respect to access, safety, parking, and drainage. After the sampling hydrants were field-verified, specific route maps were developed. Each sampling area typically contained 25 sample sites. With three sampling teams, this allowed for data collection at least every 4 hr and duplicate sampling each time a chlorine sample was collected. Sampling teams worked 12-hr shifts. The day shift consisted of three one-person teams, while the night shift consisted of three two-person teams for safety reasons. University of Cincinnati students, under the direction of CWW staff, performed all sampling for CWW.

SAMPLING RESULTS

Bulk Decay Results

Chlorine decay occurs throughout the distribution system network as the dissolved chlorine residual reacts with natural organic matter in the bulk water, pipe wall material, or the condition of the pipe wall (tubercles and biofilm). In the water quality model, values for both bulk water decay and pipe wall decay must be entered separately. Bench-scale testing for bulk chlorine decay of the finished water from both water treatment plants was performed. The resulting bulk decay coefficients for the summer months when the sampling took place were 0.19/day for the Miller Treatment Plant, and 0.16/day for the Bolton Treatment Plant.

Field Sampling Results

The field sampling program was successful in obtaining the desired information for calibration of the water quality model. These data were used to identify the proper rate coefficients for chlorine degradation during conditions representative of the sampling periods (summer maximum month conditions). Before entering the data into the water quality model, the data were reviewed relative to historical chlorine residual data collected by CWW. The objective was to confirm that the selected sample sites had indeed provided the necessary data that were representative of the entire distribution system. For this, CWW compared the 192 sample site results with over 15,000 sample results collected from 1996 to 1999. As shown in Figure 2, the summer samples collected for the water quality model were consistent with the year-round historical samples (typically less than 0.10 mg/L difference). That the water quality modeling sample results were slightly lower than the historical samples

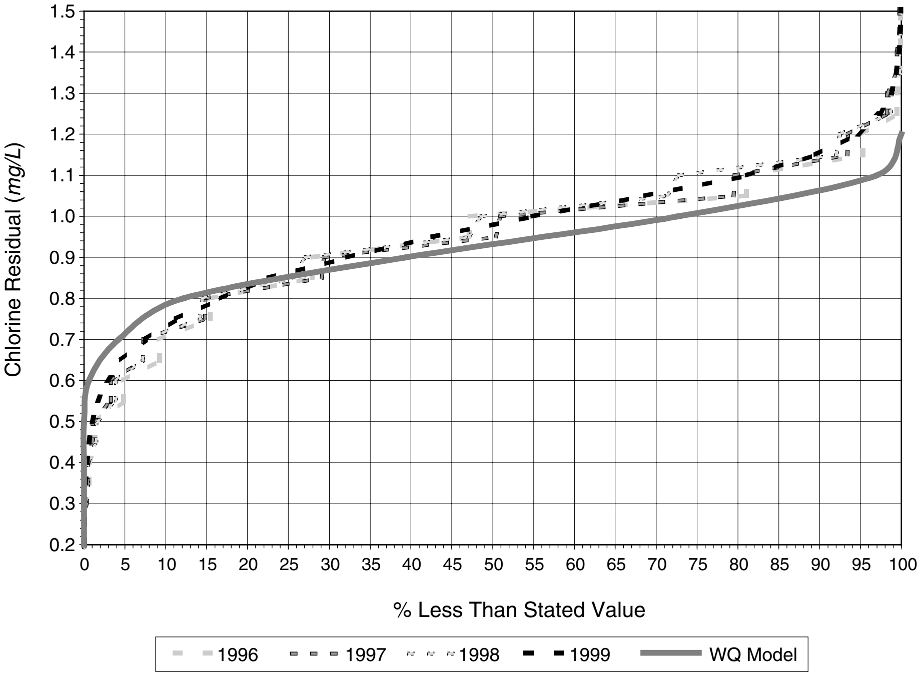

Figure 2 Historical distribution system chlorine residual versus water quality model samples

was probably due to the fact that more rigorous sites than those for the routine samples were selected for the model.

The results from the field sampling also revealed that, at least in CWW's case, no direct correlation exists between the water age and the resulting residual chlorine concentrations. In other words, the water age throughout the distribution system is not a direct indicator of the resulting chlorine residual concentration. Figure 3 illustrates this based on the samples collected. The results can be explained by the diversity of CWW distribution where parameters such as pipe size, C-factor, material of construction, and velocity probably all have an impact on chlorine decay. Consequently, the water age alone would not necessarily show much of a correlation with the chlorine residual.

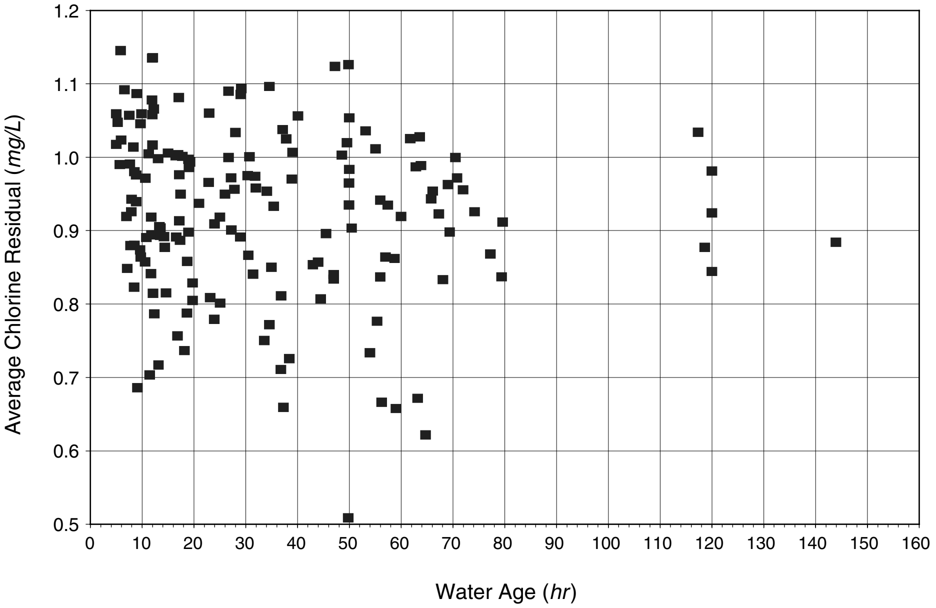

Figure 3 Distribution system sampling results versus water age

WATER QUALITY MODEL APPLICATIONS

CWW has developed 28 potential applications for the water quality model that can be organized into five categories that address water quality issues in the distribution system. The categories where the model would be most useful to CWW are:

1. To develop an increased understanding of water quality patterns and concerns in the distribution system
2. To assist with current and future water quality regulatory compliance in the distribution system
3. To provide an additional tool for evaluating operational improvements and programs that focus on continued distribution system water quality enhancements
4. To assist with the implementation of certain CIP projects
5. To assist in conducting further distribution system research

The following paragraphs describe, in general, the water quality modeling applications that CWW envisions in each of the five categories.

Understanding of the Distribution System

CWW has a large and diverse distribution system with numerous factors that could cause different rates of chlorine decay. While chlorine residual concentrations are in compliance with OEPA standards, the water quality model is a useful tool to gain a more thorough understanding of the current distribution system operation and characteristics relative to maintaining optimum chlorine residual throughout the distribution system. The first step would be to compare model-predicted chlorine residual concentrations with routine field samples in locations that are different from those selected for calibrating the model. The results will show that additional sampling or modeling may be needed on a systemwide or focused area basis. Essentially, this is an additional verification of the model with respect to predicting chlorine residual. After this, a systemwide chlorine residual study can be performed with the model to identify particular areas in the distribution system that may have low chlorine residual. Subsequent field verification of these areas would be performed, followed by further water quality modeling to identify potential causes for the low chlorine residual and to evaluate potential solutions. Other potential water quality model applications would include optimizing the chlorine dose leaving the water treatment plant, evaluating booster chlorination options, identifying additional chlorine monitoring sites in the distribution system, and assessing the impact of certain depressurization events.

Regulatory Compliance Studies

Regulatory compliance studies consider where CWW will use the water quality model focus on continued chlorine residual compliance and Stage II D–DBP Rule compliance strategies. The chlorine residual studies evaluate, for example, the impact of dead ends, excessive water age, and oversized water mains on chlorine residual concentrations. For each of these issues, the water quality model is an ideal tool to evaluate potential corrective measures such as blowoffs, flushing, and pipe looping improvements. The water quality model is also an ideal tool to evaluate the location of Stage II D–DBP Rule sampling locations, the modeling of TTHM concentrations throughout the distribution system, and the identification of potential areas of TTHM concern.

Evaluation of Operational and Program Improvements

CWW's successful unidirectional flushing program will be primarily targeted for further enhancement with the water quality model. Currently, CWW conducts flushing activities in focused areas based on the number of customers' red water complaints. The water quality model will be used as a tool to potentially provide better insights into those areas that are prone to red water. Two issues relative to red water will be investigated. First, the water quality model will be used to correlate the predicted chlorine residual concentration in specific areas prone to high red water complaints with actual field chlorine data. This type of information could indicate that areas prone to red water have higher chlorine losses than other areas in the distribution system. If this type of information can be confirmed, then the water quality model can be used as a tool to further help prioritize flushing activities as a function of customer complaints in combination with chlorine residual data. Second, the water quality model will be used to investigate if wall decay values would increase due to unlined cast-iron pipe, low C-value, smaller pipe size, velocity, and high water age. These parameters could also be directly associated with conditions that favor the creation of red water. If such a relation can be confirmed with the model and a certain level of field verification, a comparison of wall decay factors in red water districts may provide a useful tool to further help prioritize flushing activities in conjunction with customer complaint data.

Assistance With CIP Projects

The CWW distribution system has over 2,800 miles of water mains, and nearly 40% of these are unlined cast-iron pipes that are between 60 and 100+ years old. Similar to many other water utilities, CWW has an active pipeline rehabilitation and replacement program. The water quality model will provide additional information that can be incorporated into the overall ranking of pipelines that are candidates for rehabilitation or replacement. An example of how CWW plans to use the water quality model is an evaluation of the hydraulic and water quality benefits of replacing/rehabilitating water mains, particularly in areas with relatively low chlorine residual. Other applications that will provide valuable information relative to the rehabilitation/replacement program may include characterizing the impact of unlined cast-iron pipes on chlorine residual and evaluating the relationship between wall decay rates and pipe C-values. Again, these data, in conjunction with more conventional criteria, will enhance the ranking of candidate pipelines.

Distribution System Research

CWW anticipates that there will always be new water quality issues or studies in the distribution system for which the water quality model can provide valuable information. In particular, the model can be used to assess water quality issues associated with the expansion of the distribution system. Also, CWW has two water treatment plants with a floating interface in the distribution system. The model would be a useful tool to evaluate water quality impacts associated with altering the location of the interface.

CONCLUSIONS

The implementation of a systemwide water quality model has provided CWW with an additional tool to evaluate numerous water quality–related issues throughout the distribution system. The implementation, however, is a significant effort that should be carefully planned and executed to obtain a well-calibrated model that accurately represents the distribution system. Key findings presented in this chapter are summarized below:

- A water quality model should be based on an up-to-date hydraulic EPS model that accurately predicts flow, pressure, and storage tank water levels throughout the distribution system. Pilot testing of water quality modeling in focused distribution system areas can provide useful insights into basic data needs, staffing requirements, and potential model performance prior to initiating a full-scale model.
- A rational sampling plan that addresses site-specific distribution system physical and temporal characteristics should be developed to generate representative calibration data. Field verification of sampling locations prior to starting the sampling is a time-consuming but essential task to minimize the impact of field sampling on customers.
- The use of a rational sampling plan resulted in chlorine residual data that were consistent with historical data collected throughout the entire distribution system.
- No direct correlation was found between the water age and the chlorine residual in the distribution system. This suggested that numerous other factors, such as pipe material, C-value, velocity, and pipe size, have an impact on chlorine decay.

- CWW developed 28 applications for the water quality model that will provide a better understanding of water quality patterns in the distribution system and will further enhance current distribution system operations and programs.

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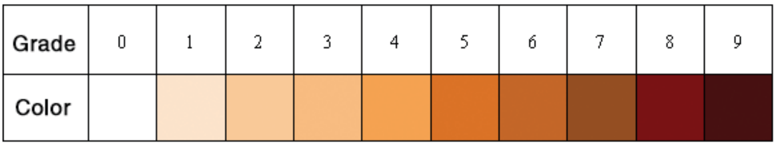
NOTE: *f.* indicates a figure; *t.* indicates a table.

Color Section

Blanks on right turn from blue to red during the course of the test while the samples on the right remain blue due to inhibition of oxidation.

Color Chart



Grade	0	1	2	3	4	5	6	7	8	9
Color	⬜	🟧	🟧	🟧	🟧	🟧	🟧	🟫	🟥	🟫

Assign a color grade by shaking the bottles in turn and comparing the resultant color with the color chart.